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COMPARATIVE ASPECTS OF FIVE PIEZOMETER DESIGNS

by

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The undersigned certify that they have read, and recommend to the faculty of Graduate Studies for acceptance, a thesis entitled "COMPARATIVE ASPECTS OF FIVE PIEZOMETER DESIGNS" submitted by DENNIS ARNOLD LINDBERG in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

This investigation is concerned with the response and sensitivity of field piezometer designs. The suitability of five selected piezometers for use in impermeable soils was studied. Two diaphragm type, two hydraulic and one open standpipe piezometer were tested in the laboratory. A test tank containing a near saturated clay with a permeability of 2.5×10^{-7} cm/sec. was employed for the tests.

Response times from one minute for the diaphragm type to approximately 4000 minutes for the open standpipe piezometer were observed. Good agreement was found in all cases between the observed response curves and those calculated from Hvorslev's theory.

It is concluded that the response characteristics of a piezometer system should be considered prior to its selection for a particular application. Hvorslev's theory can be used to advantage in assessing the relative response characteristics of various piezometer systems. By this means it is possible to select the type best suited for a given set of field conditions. Although open standpipe piezometers are simple and rugged in design they may be expected to have excessively long response times under certain conditions. From the point of view of response, diaphragm piezometers are ideally suited for pore pressure measurements in low permeability soils. There are however, few cases where the long term reliability and accuracy of field installations of such instruments have been established beyond doubt. Depending on the length and flexibility of the connecting tubing, the response characteristics of the hydraulic piezometers lie between those of the open standpipe and the diaphragm piezometer.

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TERMS AND SYMBOLS

CAB	Cellulose acetate butyrate
E.N.R.	Engineering News Record.
F	Shape factor, a constant reflecting the size and shape of a piezometer filter intake (TABLE 4.2)
G.S.C.	Geological Survey of Canada
M.I.T.	Massachusetts Institute of Technology
P.F.R.A.	Prairie Farm Rehabilitation Administration
PP	Polypropylene
PS	Polystyrene
PVC	Polyvinyl chloride
PVNC	Polyvinylidene chloride
SGI	Swedish Geotechnical Institute.
U.S.B.R.	United States Bureau of Reclamation
λ	Volume factor of a piezometer system, equal to the volume of water which must enter the system for a unit increase in pressure.

CHAPTER I

INTRODUCTION

1.1 General

Pore water pressure plays an important part in every computation concerning the stability of natural and man-made slopes, the settlement due to consolidation of silt or clay strata under load, the safety of a spillway with respect to uplift, and in many other computations pertaining to the design of structures consisting of or located above fine grained soils. In many geotechnical problems, such as in rapid construction, the pore water pressure is dependent on the magnitude of the stresses tending to lead to instability, and reasonable design estimates of the pore water pressure may be made using the results of laboratory consolidation and triaxial tests. If a major structure is being designed, subsequent field pore water pressure measurements during construction are required to verify the adequacy of the design assumptions. In other geotechnical problems, such as the stability of a natural slope, the pore pressure is an independent variable controlled either by the ground water level or by the flow pattern of impounded or underground water. With problems of this latter type, the magnitude of the pore water pressure can be determined only by field measurements.

1.2 Statement of Problem

Over the past ten years in Western Canada a number of dramatic failures have occurred in natural slopes consisting of, or underlain by, highly overconsolidated clay shale deposits (Hardy 1957, 1962, 1963). Conventional effective stress stability analyses for two of the failures (Hardy 1962), based on a comprehensive knowledge of the soil profile, field

pore pressure measurements, and triaxial tests, showed the slopes to have a substantial factor of safety against movement even though the factor of safety must have been unity at the time of failure. A similar anomaly between the known and calculated factors of safety for slope failures in overconsolidated clays has been found in England (Skempton 1964).

In most stability problems the magnitude of the actuating forces can be estimated with reasonable accuracy from a consideration of statics. Therefore, in an attempt to explain the existing anomaly, attention has been directed towards the variables controlling the shearing strength of the soil. The Mohr-Coulomb equation commonly used to express the shearing resistance of a saturated soil, in terms of effective stresses, is as follows:

$$\tau_f = c' + (\sigma - \mu) \tan \phi'$$

where τ_f = the shear stress at failure

c' = effective cohesion

ϕ' = effective angle of internal friction

σ = total stress normal to the plane of failure

μ = pore water pressure on the failure surface

Skempton (1964) advanced a residual strength hypothesis to explain the difficulties experienced in applying the effective stress stability analysis to slope failures in overconsolidated clays. This hypothesis considers that c' and ϕ' of an overconsolidated soil are a maximum, as determined by a laboratory strength test, if the soil is intact and unweathered. With time, however, as the soil becomes weathered and jointed, c' and ϕ' decrease in magnitude to approach residual values, c'_r and ϕ'_r. Hardy (1962), in a different approach, advanced a swelling pressure hypothesis whereby the Mohr-Coulomb equation is modified to include a term reflecting the swelling properties of the soil as follows:

$$\tau_f = c' + (\sigma - \mu - P_s) \tan \emptyset'$$

where P_s = is the estimated potential swelling pressure of the soil.

Both of the foregoing hypotheses are supported by a limited number of case histories (Skempton 1964, Hardy 1962), however, the possibility also exists that the true pore water pressure, μ, in the actual zone of failure has not in fact been measured. This could be due, either to the difficulty of locating a piezometer in the actual zone of failure, or to the use of a piezometer system with a high volume factor and consequently, with soils of low permeability, a long response time.

1.3 Purpose of Investigation

The purpose of this study was to review the advantages, disadvantages and limitations of various piezometer designs, to present a summary of the major factors influencing the satisfactory performance of piezometer installations, and to report on the results of laboratory response tests conducted on five selected types of piezometer systems.

1.4 Scope of Study

In the literature review, CHAPTER II, the three major types of piezometer systems are described, a historical review of the development of various types of piezometers is presented, and the major factors influencing the measurement of pore pressure, together with reported methods of estimating and reducing their influence, are summarized.

Three theories which have been developed to predict the response time of a piezometer installation, are presented in CHAPTER III. Values calculated according to two of these theories are subsequently compared with the results of laboratory response tests.

CHAPTER IV describes the piezometers tested, the test apparatus used, the properties of the soil in which the tests were conducted, and the procedures followed.

In CHAPTER V, the results of this investigation are presented and discussed. The accuracy of the Maihak piezometer is discussed, and the difficulties experienced with the Warlam piezometer are outlined. Laboratory response test results are analysed and compared with theoretical response relationships.

Conclusions and recommendations for further research are included in CHAPTER VI.

CHAPTER II

LITERATURE REVIEW

2.1 General

Since the time when engineers first began to recognize the importance of measuring insitu pore water pressures, many different types of piezometers and piezometer systems have been designed. Depending on the principle used to make measurements, all of the various designs can be classified into three broad groups: 1) Open standpipe piezometers, 2) Hydraulic piezometers and 3) Diaphragm piezometers.

The various open standpipe piezometer designs consist essentially of some type of porous intake tip, often surrounded by a filter of sand, and connected to a vertical standpipe which extends to ground level. Changes in pore water pressure acting at the location of the tip cause water to enter or leave the standpipe until such time as the water level in the standpipe is in equilibrium with the pore water pressure. A probe (Fig. 2.8) is lowered into the standpipe to determine the water level. By knowing the elevation of the tip with respect to the ground surface, the pore water pressure at the tip can be calculated.

Hydraulic piezometer systems are generally made up of a porous tip connected by either one or two tubes to a Bourdon gauge or mercury manometer. The porous tip and tubes are filled with de-aired water so that a change in pore water pressure at the tip is reflected by an equal change in pressure on the gauge. Gauge pressure corrected for the difference in elevation between the gauge and the tip is equal to the pore water pressure at the tip location. The hydraulic piezometer systems are restricted to use in embankments or shallow foundation investigations. Penman (1956) stated that in any

field installation the tubes should not rise more than 20 feet above the tip (or lowest piezometric level), and it is preferable for the Bourdon gauges to be below the tip (or piezometric level). If these conditions are not met, extreme difficulty may be encountered in keeping the system free from air.

Diaphragm piezometers, as the name implies, consist of a cylindrical shell enclosing a sensitive diaphragm at one end. One side of the diaphragm is protected by a porous stone and is subjected to pore water pressure. With one group of diaphragm piezometers, tubes lead from the tip to the surface so that the unknown pore water pressure acting on one side of the diaphragm is balanced by a known pressure on the other side, and a direct measure of the pore water pressure is obtained. With the other group, an electrical conductor connecting the tip to the surface enables the amount of deflection of the membrane under a given pore water pressure to be measured electrically. A calibration curve must be obtained for the latter type of piezometer tip prior to its field installation.

2.2 Historical Development

Although the concept of effective stress and the consequent importance of pore water pressure was recognized by Terzaghi in 1923, it was almost ten years later that the first attempt was made to measure field pore pressures.

One of the first piezometer installations was made in Holland in 1930 (Biemond, 1936) to measure pore water pressure in the clay foundation of a railway embankment. The apparatus consisted of a vertical standpipe, 1-1/2" in diameter, with the bottom end perforated and encased in a fine copper wire mesh. The standpipe was installed in a drilled hole, coarse sand was placed around the intake and a seal of clay was placed above the filter.

In 1934, the U.S. Bureau of Reclamation (U.S.B.R.) installed open standpipe type piezometers in the embankments at Hyrum Dam and at Agency Valley Dam (Daehn 1962). It was found that partial plugging of the system often occurred within a short period after installation and the response time became very large. Plugging of the system was attributed to corrosion of the metal intake and to the fact that a fairly large flow of water was required to enter or leave the system during changes in the pore water pressure.

Between 1935 and 1938, a diaphragm piezometer similar in principle to the Goldbeck-cell (Goldbeck 1938) was developed and installed in a number of dams by the U.S.B.R. (Daehn 1962). With this design the pore water pressure deflected a sensitive diaphragm sufficiently to complete an electrical circuit. Air pressure acting on the other side of the diaphragm was increased to the point where the circuit was just broken. The applied air pressure at this instant was taken to be equal to the pore water pressure. Although this design was found to give accurate readings, a number of difficulties were experienced with its use. The membrane was very delicate and could be permanently warped by faulty application of excessive air pressure by an inexperienced operator. The outer portion of the tip was subject to electrolytic and chemical corrosion, and the inner electrical contacts tended to become fouled due to condensation of moisture from the compressed air. The U.S.B.R. discontinued the use of the Goldbeck-type piezometer until 1958, when a modified version was installed in the Little Wood River Dam. Results from this latter installation have not justified further large scale use of the apparatus (Daehn 1962).

Ringeling (1936) recognized that in order to measure true pore water pressures a piezometer system should have a very small volume factor, i.e., it should be able to measure changes in pore pressure with a minimum flow of water into or

out of the system. His piezometer design consisted of a 3/4 inch pipe with a perforated intake centrally located in a sand filter, in a bore hole. A clay seal was placed above the filter and the pipe, filled with water, was connected to a capillary mercury-gauge. This design was the forerunner of the present day hydraulic type piezometer.

In 1939 the U.S.B.R. designed a twin tube hydraulic piezometer system to measure hydrostatic potentials within their earth embankments (Daehn 1962). Over the period of years since 1939, various types of tubing, tip material, and porous stones have been tested. Evolving from a vast amount of experience, the present design (Fig. 2.1 and 2.2) consists of a high air entry porous stone pickup sealed in a small polypropylene tip, with two saran (polyvinylidene chloride) tubes leading from the tip to a gauge house.

Terzaghi (1943) analyzed the various pore water pressure measuring devices in existence at that time and concluded that none of the designs were suitable for measuring pore water pressures for deep foundation type investigations in low permeability soils. As a consequence, he designed a diaphragm type piezometer (Fig. 2.3) in which the deflection of the diaphragm was measured with a Carlson elastic-wire strain meter and related to pore water pressure by means of a calibration curve. After several satisfactory field tests, permanent installations of this piezometer were made beneath the ore yard of a steel plant in the U.S.A. Within a period of two years, all of the diaphragm piezometers became defective and were replaced by hydraulic type piezometers (Terzaghi 1960).

Speedie (1948) described a Goldbeck-type diaphragm piezometer which was modified to such an extent that it gave consistent readings over 3-1/2 years of records in the Eildon Dam, Australia. The modified design included the following features: 1) use of dissimilar metals was avoided, 2) air was dried through calcium chloride filters prior to taking a

reading and 3) the tip was provided with an extra tube so that any condensed moisture collecting in the tip could be blown out (Fig. 2.4).

In the Netherlands, Boiten and Plantema (1948) designed a diaphragm piezometer which could be pushed into the soil. This design consisted of a sensitive diaphragm whose deflections under pressure were measured with electrical resistance strain gauges. In the original design the diaphragm was not located in a neutral position in the tip. As a result the earth pressure acting on the tip created a questionable influence on the pore water pressure measurements. This was corrected in a later design (Plantema 1953).

Since 1948, the Swedish Geotechnical Institute (S.G.I.) has done extensive research in developing a pore water pressure measuring system which would be rugged, sensitive, and have a small response time (Kallstenius 1956, 1957). Their unique design uses a standpipe with a filter tip which can be forced into the soil to a desired location. The standpipe and filter are so designed that interchangeable hydraulic or diaphragm type piezometer tips can be lowered into the standpipe and connected to the filter after the standpipe is in place. This has the advantage that the piezometer tip can be removed and recalibrated or replaced by a more suitable tip at any time.

A typical S.G.I. filter standpipe is shown in Fig. 2.5. Fig. 2.6 (a) shows one of the S.G.I. oil filled hydraulic piezometer tips. The tip is connected to a Bourdon gauge by a single small diameter copper tube. The tube and upper portion of the tip are filled with silicon oil, the oil being kept separate from the pore water by a thin flexible rubber diaphragm located in the tip. With this system, no difficulties are encountered with de-airing or freezing, and since the specific gravity of the silicone oil is less than

that of water, it permits measurements of ground-water tables which are slightly deeper than if the line was filled with de-aired water.

The S.G.I. electro pneumatic diaphragm piezometer (Fig. 2.6 (b)) incorporates two brass bellows in its design. The free ends of the bellows are capped by platinum contact surfaces which, under conditions of atmospheric pressure, are separated by a fixed distance. Pore water pressure acting on the lower bellows causes it to deflect and decrease the spacing between the bellow contact surfaces. When a reading is desired, air pressure is applied to the upper bellows, deflecting it until it makes contact with the lower bellows as indicated electrically by use of a milliammeter. The air pressure required to make contact is related by means of a calibration curve to the pore water pressure.

Casagrande (1949), faced with the problem of measuring pore water pressures at a location which precluded the use of hydraulic type piezometers, developed a non-metallic open standpipe piezometer system incorporating a large filter intake, a small diameter standpipe, and a positive seal above the filter (Fig. 2.7). The combination of a small diameter standpipe and a large intake area with the relatively high coefficient of permeability of the soil in which the tip was first installed, 1×10^{-6} cm. per sec., contributed to the piezometer's satisfactory performance in its first application (Gould 1949).

With the typical Casagrande piezometer installation, the standpipe is installed vertically and an electric probe, such as shown in Fig. 2.8, is used to determine the water level. When the Casagrande piezometer is installed below or in an embankment prior to completion of the embankment, the vertical standpipe often is a hindrance to construction equipment. To avoid this, modified Casagrande type piezometers (Fig. 2.9) were installed on the Swift Hydroelectric Project,

Washington (de Luccia 1958). The standpipe was placed vertically for a certain length above the tip and then brought horizontally to a convenient location. A small tube was installed in the standpipe, extending the full length of the standpipe. Pore water pressure measurements were made by slowly introducing air into the small tube. When air just started to bubble around the end of the small tube, the air pressure was assumed equal to the pore water pressure. One obvious disadvantage of this system is that any water initially in the small tube is pushed either into the formation or into the standpipe and consequently, unless time is allowed for equalization, the pore water pressure readings will be too high.

Prior to 1950, very few piezometer installations had been made in Great Britain (Penman 1956). Open standpipe piezometers had been used to measure pore water pressures in the Knockendon Dam in 1944; however, it was not until the construction of the Daer and Usk dams in 1950 that the Building Research Station developed their first hydraulic piezometer systems. The initial design was very similar to that of the U.S.B.R., but recently a high air entry tip, suitable for use in partially saturated soils, has been designed at Imperial College (Fig. 2.10), (Bishop, Kenard and Penman, 1961).

In 1951, the Maihak diaphragm piezometer, manufactured in Germany, was brought onto the market (Muhs, 1954). The Maihak piezometer tip houses a sensitive diaphragm, connected to which is one end of a pretensioned wire (Fig. 2.11). Pore water pressure causes the diaphragm to deflect and thus the tension in the wire is reduced. This in turn alters the natural frequency of vibration of the wire. To obtain a measure of the pore water pressure, the wire in the piezometer tip is excited to damped natural vibrations by an impulse from a receiver. The receiver is tuned in to give a reading of the natural frequency of the wire, and this reading is converted to pressure by means of a calibration curve.

About 1952, a French vibrating wire, diaphragm piezometer was designed (Fig. 2.12). The Telemac design is similar in principle to the Maihak but uses a sensing tube rather than a sensitive diaphragm. A pretensioned wire is connected to the ends of the tube which acts as a spring and compresses under the application of pore water pressure. A measuring set monitors the change in natural frequency of the wire and these changes are related to pore water pressure.

Vourinen (1957, 1961) described a diaphragm piezometer, still in the development stages in Finland, which used an inductance coil system to measure the deflection of the diaphragm (Fig. 2.13). This piezometer contains two fixed coils, and one moveable coil which follows the deflection of the membrane. The mutual inductance between the coils changes with a change in position of the moveable coil and is related to pore water pressure by means of a calibration curve.

The Norwegian Geotechnical Institute (Bjerrum 1961) developed an open standpipe piezometer (Fig. 2.14) consisting of a porous metallic tip which can be coupled to a steel "E" size drill rod and pushed into the soil to the desired elevation. A small diameter polyethylene tube is connected through the drill rods to the porous tip.

Three air operated diaphragm piezometers have recently come onto the market in the United States; the Warlam, the Hall, and the Dames and Moore piezometers. The Warlam piezometer (Fig. 2.15) is basically a sensitive diaphragm type check valve, shut tight by external water pressure, and opened up by internal air pressure (Warlam 1963). The tip has an air entry line connected through a pressure regulator to an air pressure supply, and an air return line terminated in a container of water, called an air flow indicator. To take a pore pressure measurement, air pressure on the inside of the tip is increased until the check valve just opens, as indicated by air bubbles rising in the air

flow indicator. The air pressure is slowly decreased until air bubbles no longer appear in the indicator, at which time the air pressure is equal to the pore water pressure.

The Hall piezometer, recently installed in the Briones Dam (E.N.R., 1964), is not unlike the Warlam piezometer in that the tip contains a collapsing bellows, and is connected to the surface by an inlet and outlet tube. Nitrogen is used to balance the pore water pressure acting on the bellows. As a balance is approached, bubbles escape into the return line. When these bubbles appear at a certain speed and amount, the applied nitrogen pressure is assumed to be equal to the pore water pressure and a reading is taken.

The Dames and Moore diaphragm piezometer (Fig. 2.16) uses a double reinforced neoprene diaphragm 1/8 inch thick which spans a small nozzle-shaped opening. Two small tubes inside a larger one connect from the measuring side of the diaphragm cavity to the surface. A pressure gauge is connected to one of the small tubes while air is bled into the other. As the air pressure is increased the diaphragm is moved away from the nozzle and excess air escapes to the surface through the large tube. When the diaphragm cavity pressure, as determined by the pressure gauge, stops increasing, the pore water pressure is equal to the gauge pressure.

Brooker (1965), at the University of Alberta, designed a diaphragm type piezometer, Fig. 2.17, which incorporates a Dynisco APT 25 pressure transducer to measure pore water pressures. This piezometer is presently under construction. It is expected that it will be subjected to laboratory tests and subsequent field trials during 1965.

2.3 Factors Influencing Pore Pressure Measurements

There are a number of factors which may influence the accurate measurement of pore water pressures. Some of these affect all types of piezometer installations equally;

others have a greater effect on a particular type of piezometer and may greatly influence the choice of piezometer system for any given location. The following subsections review the major factors which may affect a piezometer installation.

2.3a Installation Time Lag

All of the various methods of installing foundation type piezometer tips tend to alter the stress conditions in the soil and hence disturb the natural pore water pressure in existence prior to the installation. The effect of installing a piezometer may be to increase or decrease the pore pressure, but either case, a certain period of time must be allowed for this pore water pressure change to dissipate. The time required, termed the installation time lag, depends on the method of installation, the dimensions of the zone of stress change, and the permeability, sensitivity to disturbance and consolidation characteristics of the soil (Hvorslev 1951).

The Swedish Geotechnical Institute follows a procedure of pushing the piezometer tip axially into the ground (Kallstenius 1956). In a saturated clay, the disturbance and displacement of the soil during the driving of the tip gives rise to increased pore pressures. The magnitude of these increased pore pressures has been estimated and measured by the Swedish Geotechnical Institute to be in the order of 8 to 10 times the shear strength of the soil. Installation time lags ranging from a few minutes to two weeks have been measured.

Terzaghi (1943) ran some tests whereby a diaphragm piezometer tip was pushed 6 inches and then a further 6 inches into the bottom of a drill hole in homogeneous soft clay. When the tip was pushed in the first 6 inches, an excess pore pressure of 35 feet of water was recorded. After this had dissipated to essentially a constant value, the tip was pushed a further 6 inches. In the latter case an excess pore pressure

of 98 feet of water was measured, which after 6.7 hours had decreased to 10 feet excess pressure.

In other than soft deposits it is impossible to push a piezometer tip into the ground, and the common procedure is to place and seal the piezometer in a borehole. The removal of soil with the advancement of the borehole causes a reduction in the stresses in the vicinity of the bottom of the hole. As a result, the soil tends to swell with an associated decrease in pore pressure. (Hvorslev 1951). An empirical relationship which has been used as a guide in estimating installation time lags associated with borehole installations is that time lag in months is equal to 0.2 divided by the coefficient of swelling of the soil in feet^2/month (Vaughan 1964).

Installation time lag is of most significance when only a single or a few observations are to be made at each depth and location, as is often the case with the driven piezometer tips. Suggested ways of reducing the installation time lag are to reduce the dimensions of the piezometer tip and/or borehole, and to initially apply a head to the piezometer tip in such a manner as to speed up equalization (Hvorslev 1951).

2.3b Measurement Time Lags

The measurement of pore water pressure with any piezometer system requires a certain amount of energy; this energy being proportional to the volume factor of the system, i.e., to the volume of water which must flow to or from the system for a unit change in pressure. The response of a pore pressure measuring system is therefore delayed by the time taken for this required flow. The response time or measurement time lag of a piezometer installation is not only affected by the system's volume factor, but also by the size and the shape of the intake, and by the permeability and

compressibility of the soil in which measurements are to be taken (Gibson 1963).

In selecting a piezometer system for the measurement of pore water pressures, particularly if the observations are to be made in a low permeability soil, the response time of the system deserves careful attention. If the system has a long response time and fluctuating insitu pore pressures are to be measured, the observed readings will be out of phase with true insitu pressures. Also, the plot of observed pore pressures versus time will exhibit a smaller amplitude than actually occured insitu.

CHAPTER III of this report outlines theories which have been proposed, by means of which measurement time lags can be calculated for various conditions. These theories are based on a number of limiting assumptions; however, the available laboratory and field data indicate that they may be of value, at least in indicating the relative response characteristics of various types of piezometers.

Penman (1961) conducted a series of laboratory response tests on selected open standpipe, hydraulic and diaphragm piezometers. He showed that measured times to reach equilibrium with a pore pressure change were of the same order as those calculated by Hvorslev's theory. Calculated times to reach intermediate degrees of equalization did not however, agree with measured times. Bazett (1959) reported laboratory tests on a modified U.S.B.R. hydraulic piezometer which indicated a response time in the order of 20 minutes. The theoretical response time was calculated to be 20 minutes.

A field response test on a Casagrande piezometer installed in soft Boston Blue clay ($k_H = 1.2 \times 10^{-6}$ cm/sec, $k_v = 3.2 \times 10^{-8}$ cm/sec) showed the time required for 99 percent response to be 4-1/2 hours (Gould 1949). This was in close agreement with the theoretical time for 99 percent response of 5 hours as given by Hvorslev (1951).

Skempton and Henkel (1961) studied the response characteristics of a Casagrande type piezometer (an 18 inch long by 4 inch diameter intake connected by a 3/8 inch I.D. standpipe) installed in London clay, with a permeability of 2.2×10^{-9} cm/sec. The measured response after 40 days was approximately 89 percent. They show that a theoretical response curve calculated by Hvorslev's theory agrees very well with observed pore pressure readings.

Field measurements of the response times of Building Research Station type hydraulic piezometers, given by Bishop, Kennard and Penman (1961), were compared with values calculated using Hvorslev's theory (Penman 1961). The permeability of the soil was in the order of 1×10^{-8} cm/sec. The Building Research Station disc type piezometer connected to a manometer by 540 feet of polythene tubing, had a measured time for 90 percent response of 1000 minutes compared to a theoretical time of 885 minutes. The Bishop type high air entry piezometer tip connected to a similar manometer by 900 feet of nylon tubing, had a measured t_{90} of 180 minutes and a calculated value of 134 minutes.

Vaughan (1964) stated that limited field tests have shown a good fit with Gibson's theory (Gibson 1963) for response time. He commented further that at high degrees of equalization, there is little difference between Hvorslev's and Gibson's theories.

2.3c Seals

When a foundation piezometer is installed, care must be taken that there is no communication between the zone in which measurements are to be taken and zones of higher or lower pore pressure located above this. If the piezometer tip is pressed axially into the ground there is little danger of leakage (Kallstenius 1956). If however, the piezometer tip is located in a sand filter at the bottom of a drill hole, the space between the piezometer tubes and the wall of the hole or

casing must be carefully sealed. Ideally, the seal should have the same properties as the soil in which it is being placed, i.e., similar permeability, compressibility, consolidation and swelling characteristics.

Vaughan (1964), by making simplifying assumptions as to geometry, obtained a theoretical solution for the measurement error of a bore hole piezometer as a result of the permeability of the seal being larger than that of the surrounding soil. He concludes that for an error of 5 percent, the ratio of seal permeability to ground permeability in a 6 inch diameter borehole can be 7 for a 5 foot, 50 for a 15 foot, and 300 for a 50 foot deep hole.

Over the years many different types of seals have been investigated and used. Terzaghi (1943) suggested the use of a 2 foot seal of portland cement mortar, the remainder of the hole being filled with a heavy thixotropic clay slurry. He stated that the clay slurry should be less permeable than the surrounding soil and should stiffen up to a certain extent without becoming rigid.

In a later paper, Terzaghi (1943) considered in some detail the problem of seals and borehole fillers for piezometer installations in a plastic clay. He stated that any clay or bentonite slurry, in a workable consistency, is very permeable compared to an insitu plastic clay; and therefore, the only material which can be relied upon to form a seal is cement with a low water-cement ratio. Recognizing the undesirable effect of having a rigid column of cement extending from the piezometer tip to ground surface, he suggested that an 18 inch thick, very compressible cushion of mica slurry be introduced at 10 foot intervals. A low heat cement mixed at a water-cement ratio of 0.4 by weight, and a mica slurry prepared by mixing one part of minus 200 mesh mica with

1.3 parts of water by weight, were recommended. To form a firm base for each layer of cement, it was suggested that a 6 inch thickness of sand and pea gravel be placed above each mica cushion.

Casagrande (1949) developed and successfully used a borehole seal composed of alternate layers of sand and bentonite. With the Casagrande sealing procedure, a 3 foot layer of sand is placed above the piezometer filter and tamped to minimize the effect of swelling pressures of the overlying bentonite. Next five 3 inch layers of bentonite balls are placed, each being tamped in a prescribed manner. This is followed in turn by a 2 foot layer of tamped sand, another bentonite seal, and a further 2 foot layer of sand. The bentonite balls are formed to a 3/8 inch diameter from bentonite which has been prepared at a water content slightly greater than the plastic limit.

The Soil Mechanics and Materials Division of the Prairie Farm Rehabilitation Administration (PFRA 1964) have found that they can greatly speed up the Casagrande procedure of sealing a borehole piezometer by using bentonite doughnuts rather than balls. To prepare the doughnuts, they mix 4 parts of bentonite (liquid limit over 500 percent and plastic limit approximately 35 percent) as thoroughly as possible with 3 parts of water, and compact this in a Proctor mold using approximately standard Proctor procedure. A 1/2 inch pipe is used to form a hole through the center. With the piezometer intake and filter sand in place, the doughnuts are slid down the piezometer leads and compacted to form a tight seal. Bentonite doughnut seals, with a sulphate resistant cement grout backfill, have been used in boreholes up to 100 feet deep. Beyond this depth, cement grout has been used both for a seal and for backfill.

A simpler procedure for placing a bentonite seal, when there is water standing in the borehole, is to fill

ladies' nylon stockings with dry bentonite, drop them down the hole, and subject them to compaction. Vaughan (1964) reported that one such seal was excavated sometime after installation and was found to be satisfactory. This sealing procedure was used in a recent landslide investigation just east of the city of Edmonton with apparent success (Painter 1965).

Lambe (1959, 1960), in an effort to decrease the time required to seal a Casagrande piezometer, investigated the use of a clay chemical grout as a seal. With Lambe's procedure, a one inch bentonite seal is placed above the filter to prevent invasion by the grout. Then a predetermined amount of slurry of caly and catalyzed AM-9 chemical* is poured in. Polymerization occurs in 15 to 40 minutes to give a strong, rubbery, impermeable seal. Tests run in the Soil Engineering Laboratories at M.I.T. showed the seal to be equal to or superior to the bentonite seal in all respects. Karol (1960) reported that, with careful control of gel time and viscosity, AM-9 solution can, by itself, be used as a piezometer seal with virtually no danger of grout penetration into the piezometer intake. The Cyanamid of Canada Limited Chemical Grout Field Handbook lists the permeability of AM-9 gel as 10^{-10} cm/sec and states that no syneresis has ever been observed. Under dry conditions the gels will however, dehydrate and shrink.

An oil based chemical grout, Polythixion, has recently been recommended as a borehole sealant (Robertshaw, 1964). Robertshaw stated that this material does not dry out or shrink and has a permeability of 1×10^{-9} to 1×10^{-11} cm/sec. As an alternative he suggesed the use of a mixture of 1-1/2 parts by weight of bentonite, and 1 part of ordinary Portland

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* Trademark of American Cyanamid Company

cement. This grout has an indicated permeability of 10^{-6} to 10^{-8} cm/sec.

2.3d Protection from Freezing

If an open standpipe or hydraulic type piezometer system is installed in an area where below freezing temperatures prevail for part of the year, some steps must be taken to avoid freezing of the system. For the majority of hydraulic piezometer systems, especially those installed in earth embankments, the leads are buried below the frost level and are brought to a common, heated gauge house. There are numerous occasions however, where it is not feasible to provide a heated gauge house, and some low freezing point fluid must be used in the piezometer leads.

Hvorslev (1951) suggested that inactivation and damage by frost could be prevented by filling the upper part of a piezometer system with kerosene or some other oil. If this is done, the position of the oil-water interface must be known and the difference in specific gravity between the fluids must be taken into consideration. Hvorslev noted that if observations are extended over a long period of time, it may be difficult to keep track of the position of the oil-water interface. Also, if the interface is located in a piezometer tip with very fine pores, considerable errors may be caused by menisci formed in the pores, and by the difference in surface tension of the water and oil.

Bozozuk (1960) stated that an antifreeze solution can be used to prevent freezing in open standpipes, with the specific gravity of the antifreeze being taken into account in calculating pore pressures. Eden (1961) reported the use of an ethylene glycol solution, with a specific gravity of 1.04, in both open standpipe and hydraulic piezometers. Soderman (1961) presented the details of a foundation installation of a hydraulic piezometer system which operated

successfully to temperatures of -40°F. The upper part of the piezometer system, to a depth of 1-1/2 times the frost depth, was filled with a solution composed of 4 parts methanol, 2 parts glycerol, and 3 parts water by volume. A 3-inch oil seal was used to separate the antifreeze from the water filling the rest of the system. The antifreeze solution had a specific gravity of 0.991 at 78°F and a freezing point of -60°F.

It has been recognized that certain difficulties may be encountered by the use of an antifreeze solution in a piezometer system. Soderman (1961) stated that ideally the piezometer fluid should consist of an aqueous solution similar in all respects to the insitu pore fluid of the soil in which the piezometer is placed. If a fluid is used that has an ion concentration different from that of the insitu pore fluid, osmotic pressures may develop and result in erratic pore pressure readings. An example of such a situation is reported by Daehn (1962) where osmotic pressures as high as 630 psi were measured, when the fluid filling a hydraulic piezometer system consisted of a 50 percent ethylene glycol-water solution. Daehn also reported that in several of the installations a heavy floc precipitated from some of the glycol solutions and ultimately plugged a number of piezometer tips.

2.3e Gas Bubbles

Air and other gases are often entrapped in the pores of the soil even when the soil is below the ground water table (Hvorslev 1951). In some instances, by poor choice of the material forming the piezometer tip, gases are generated in the vicinity of the tip by electrolysis (Weber 1959). The presence of undissolved gases in the soil voids has the effect of decreasing the permeability of the soil and thereby increasing the measurement time lag of the piezometer system.

With an open standpipe piezometer, any air entering the pickup will dissipate without providing a true measure of the insitu pore pressure (Daehn 1962). For this reason, Muhs (1954) considered that open standpipe piezometers can only be used in saturated soil. If, in an attempt to decrease the measurement time lag, a very small diameter standpipe is used, there is a danger that air bubbles will collect and fill the entire cross-section of the stand pipe, and false pore pressure readings will be obtained.

Accurate pore pressure observations with a hydraulic piezometer system are dependent on the system being free from air. The accumulation of air in the system affects observations in two ways. Since air has a negligible unit weight in comparison to water, corrections applied to the gauge readings for the elevation head between the piezometer tip and the gauge will be in error by an amount equal to the unit weight of water times the net cumulative vertical length of the air bubbles. Also, due to the high compressibility of the air, measurement time lags may be greatly increased, e.g., the volume factor of a piezometer system is increased by 2.6×10^{-4} cm^5/gm for 1 cm^3 of gas at atmospheric pressure (Penman 1961).

Penman (1956) suggested that the pore pressure should be read independently on each of the two piezometer leads, any difference in pressure indicating the presence of air in the system. He noted that this procedure is not infallible since, on occasions, even though both leads have given identical readings, air has been found in the system and must have filled both leads to the same extent.

Maintaining a saturated system can become extremely difficult if measurements are to be taken in partially saturated soil, or if negative pore water pressures are to be observed. To overcome this problem, hydraulic piezometer tips have been equipped with high air entry stones (Bishop, Kennard,

Penman 1961, Daehn 1962), and the use of special nylon tubing which is essentially impervious to air has been recommended (Penman 1958).

Periodic circulation of de-aired water through the piezometer leads is required to remove any entrapped air. With long lengths of small diameter tubing fairly high heads are required to circulate the water, and this tends to disturb pore pressures in the soil. The required head can be reduced, and good results have been obtained, by applying a vacuum to the return lead (Peters 1958). Little and Vail (1961) reported a technique whereby the optimum de-airing pressure was calculated so as to reduce the disturbance of pore pressures to a minimum. Penman (1956) stated that periodic field response tests are run on hydraulic piezometer installations to determine the time which must be allowed to elapse, subsequent to de-airing, before accurate readings can be again taken.

2.3f Tubing

In many cases, the properties of the tubing used with open standpipe, hydraulic, and air operated diaphragm piezometers may greatly affect the satisfactory performance of the installation. Penman (1961) reported, for example, that by adding 1000 feet of polythene tube to a hydraulic piezometer tip, the measurement lag time was increased from 13 minutes to 661 minutes. Daehn (1962) gave an example where the water loss through the walls of hydraulic piezometer tubing was so large as to result in pore pressure readings which were unreasonably low.

A great number of different types of tubing have been considered for use in piezometer installations. These include copper, polyvinylidene chloride (PVNC,SARAN), polyvinyl chloride (PVC), nylon, polythene, polythene coated nylon, polyethylene, cellulose acetate butyrate (CAB),

polypropylene (PP) and high-impact polystyrene (PS). Depending on the specific conditions under which the tubing is to be installed, a number of the following properties of tubing may have to be considered: volume factor, water permeability, air permeability, water absorption, brittleness, burst strength, collapse strength, resistance to cutting and crushing, and resistance to the flow of water (for de-airing purposes).

For open standpipe piezometers, it is desirable to use as small a diameter tubing as possible to reduce measurement time lags. The diameter commonly used is 3/8 to 1/4 inch I.D., this being governed by the diameter of the probe, and by the necessity to avoid trapping air bubbles.

The choice of tubing for use with hydraulic piezometer systems has been given much attention. Copper tubing was used in the initial U.S.B.R. design; however, during World War II, there was a shortage of copper tubing and a substitute tubing had to be used (Daehn 1962). The U.S.B.R. tried in turn, PVNC, CAB and low density polyethylene, and finally reverted to the use of PVNC tubing. On the initial installation using PVNC tubing, they found that it became brittle with age and with cold temperatures. The CAB tubing was found, after installation, to have a high water loss, 0.005712 in^3/ft/day, which led to erratic pore pressure readings. Selected because of its low rate of water absorption and low water vapour transmissibility, the low density polyethylene tubing was found to be too soft and was easily damaged by cutting and crushing. The PVNC tubing, which is presently specified for use with U.S.B.R. hydraulic piezometer tips, though it tends to become brittle at low temperatures, does exhibit low water loss and high crushing and burst strength characteristics. Recently the use of oriented polypropylene tubing, which appears to have the desirable characteristics of and is workable at lower temperatures than PVNC, has been considered.

The Building Research Station found that repeated de-airing was required for hydraulic piezometer systems connected with polythene tubing, when the gauge house or sections of the tubing were located above the piezometric level. A series of tests were therefore conducted on 3 mm. bore by 1 mm. wall polythene and 2.8 mm. bore by 1 mm. wall nylon tubing. From these tests it was found that the two types of tubing had the following properties:

	POLYTHENE	NYLON
Volume Factor	0.0598×10^{-4} cm^5/gm/ft	0.0146×10^{-4} cm^5/gm/ft
Water Loss	Nil	1.4×10^{-4} cc/hr/ft
Permeability to air	0.04 cc/meter/hr/Kg/cm^2	Impermeable

Where pressures less than atmospheric were anticipated, the use of nylon tubing was recommended (Penman 1958,1961). Many present day installations are being made with a polythene coated nylon tubing which is impermeable to both air and water and does not become brittle with age and low temperature (Robertshaw 1964).

International Power and Engineering Consultants Ltd. ran comparative tests on several types of tubing, and selected polyvinyl chloride tubing as being the most suitable for hydraulic piezometer installations in the Portage Mountain Dam. This tubing was selected on the basis of its low permeability to air, strength, and toughness characteristics (Naylor, 1964).

Bazett (1961) found that by replacing polythene with copper tubing, the volume factor of the U.S.B.R. hydraulic piezometer system could be significantly reduced. The measured volume factor of the copper tubing was 0.00338×10^{-4} cm^5/gm/ft.

The Swedish Geotechnical Institute, after experiencing difficulties with gas bubbles, have incorporated copper tubing in their hydraulic piezometer systems (Kallstenius 1956). Tests to determine the water permeability of PVC, polyethylene, and PVNC tubing were run which showed the PVC and polyethylene tubing to be permeable and the PVNC to be impermeable.

2.4 Field Performance

It is a well documented fact that the Casagrande type open standpipe and the twin tube hydraulic type piezometers, once carefully installed, are robust and have a long life (Daehn, 1962, Peters, 1958, Cooling, 1961). Diaphragm type piezometers, which have the advantage of having very small response times, do not have such an enviable record.

The U.S.B.R. experienced unsatisfactory field performance with the Modified Goldbeck piezometer, and have discontinued its use (Daehn 1962). Terzaghi designed and installed a number of diaphragm type piezometers in the fall of 1943. Two of these were damaged during installation and the remainder became defective by the summer of 1945 (Terzaghi 1960). Bazett (1959) reported the failure of Plantema piezometer tips less than one year after installation.

Muhs (1954)* reviewed the advantages and disadvantages of various types of piezometer designs and concluded that the Maihak was far superior to any other type of piezometer. He considered the Plantema piezometer unsuitable for long term measurements since the resistance strain gauges soon showed signs of age and were very sensitive to moisture. A further disadvantage of the Plantema design was that pore pressure readings were affected by the resistance of the connecting cable which in turn varies with the length and temperature of the cable. Muhs also noted that pore pressures measured

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* This paper was supplied by Maihak.

by a Telemac piezometer were influenced by the length of connecting cable, earth pressures acting on the tip and temperature fluctuations.

Le Moigne (1958) noted that the Telemac piezometer had been redesigned so that pore pressure readings were no longer affected by earth pressure or by temperature fluctuations. He pointed out that there was little difference between the Maihak and the Telemac designs, other than in the pressure monitoring equipment and in the price - the Telemac being more expensive. Of the diaphragm type piezometers, Le Moigne indicated a preference for the Maihak design.

One of the first field installations of Maihak piezometers was made, in 1952, in an earth dam at Rosshaupten, Germany. Treiber (1958) reported that installation of the tips required great care, and until experience was gained, installation costs were high. Six failures occurred during construction due to damaged cables; however, the remaining fourteen tips gave reliable readings during four years of observation.

Nineteen Maihak and fifty-eight U.S.B.R. type hydraulic piezometers were installed from 1956 to 1958 in the Eucumbene Dam in Australia. During construction, six Maihak and nine U.S.B.R. piezometers failed completely due to damaged cables and tubing. One tube of twelve other U.S.B.R. piezometers became blocked. Difficulties experienced initially in obtaining a reading from the Maihak type were considered to be due to interference by stray earth currents, and were overcome by using an isolating transformer between the gauge and the receiving instrument. Subsequent pore pressure readings obtained from the Maihak tips were erratic and did not agree with those obtained from hydraulic piezometers located in the same vicinity. As a result, little confidence was expressed in the pore pressures indicated by the Maihak tips. In an effort to determine the reason for the discrepancy between pore pressure readings obtained from

the two types of piezometer installations, Moore conducted a series of large scale laboratory tests. Under laboratory conditions, it was found that the readings of pore pressure by the hydraulic piezometer and by the Maihak gauges were in close agreement. It was further noticed that the use of an isolating transformer altered the zero readings of the Maihak tips by as much as 40 feet of water.

In the discussion of Moore's paper, Jones (1960) noted that only two permanent failures had occurred out of 237 hydraulic piezometer tips installed in six dams in New Zealand. McConnell (1960) commented that Maihak piezometers installed in Tooma dam were giving satisfactory readings. Speedie (1960) stated that he had found the Maihak piezometer to be more accurate and reliable than the hydraulic types. At Eldon it had been necessary to include filters in the Maihak circuit to eliminate interference by stray current.

The reasons for the troubles experienced by Moore (1960), with the Maihak piezometers have been explained by the Maihak Company as follows (Maihak 1965). The Maihak tips installed in the Eucumbene Dam were constructed with their negative pole connected to the piezometer casing so that the return lead of the tip was actually grounded in the soil. Stray currents from heavy construction machinery grounded nearby would therefore appear as interference on the receiving instrument, and make it very difficult to obtain a reading from the Maihak tips. Maihak considered that the use of the isolating transformer to filter out the stray currents resulted in the tip being subjected to intolerably high voltages, which affected the measuring reliability and zero stability of the tip. The design of the tip has since been improved so that the negative pole is no longer connected to the body of the tip. The present design also includes an over-voltage protection diode.

Maihak also stated that their piezometer can be calibrated to measure negative as well as positive pore pressures if this is specified in the order.

Bishop hydraulic and Maihak type piezometers have been installed in an earth dam at Diddington, Bedford, England; however, there is no literature available as yet on the comparison of pore pressures obtained.*

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* Personal communication with W. Painter

CHAPTER III

THEORY

3.1 General

A number of authors (Hvorslev 1951, Kallstenius 1956, Gibson 1963) have developed theoretical relationships which enable calculation of the response time of a piezometer installation. The following sections of this chapter give a summary of these various theories.

3.2 Hvorslev Theory

Hvorslev (1951) considered the case of a simple open standpipe piezometer, where the ground water level was constant but at time zero was not in equilibrium with the water level in the standpipe. He derived an equation for the equalization ratio based on the following assumptions:

1. The soil is fully saturated, isotropic and infinite in extent.
2. No swelling or consolidation of the soil occurs.
3. Water is incompressible.
4. Darcy's law is valid.
5. Hydraulic losses in the piezometer system are negligible.
6. The volume factor of the piezometer system is a constant.

Hvorslev considered the conditions (Fig. 31.) where at time zero the out of balance is H_o and at some other time t it is H, where $H=H_o - y$. The rate of flow of water into the piezometer at time t is given by,

$$q = FkH \tag{1}$$

where q = flow, cm^3/sec.

F = shape factor, cm, (depends on the size and shape of the intake, see Table 4.2)

k = permeability of the soil, cm/sec.

H = head, cm of water.

The volume of flow into the tip for a given time interval can be equated to the rise in the standpipe water level.

$$q\,dt = A\,dy \tag{2}$$

where dt = time interval, sec.

A = cross sectional area of standpipe, cm^2

dy = rise in standpipe water level, cm.

Combining equations (1) and (2)

$$\frac{dy}{H} = \frac{Fk}{A}\,dt \tag{3}$$

The total volume of flow required for equalization of the pressure difference, H, is V=AH. By defining the basic time lag, T, as the time required for equalization of this pressure difference when the original rate of flow, q=FkH, is maintained,

$$T = \frac{V}{q} = \frac{A}{Fk} \tag{4}$$

Equation (3) can then be written

$$\frac{dy}{H} = \frac{dt}{T} \tag{5}$$

By putting $H=H_o - y$ and applying the boundary condition that $y=0$ for $t=0$, equation (5) can be integrated to give

$$\frac{t}{T} = \ln\frac{Ho}{H} \tag{6}$$

or

$$\frac{H}{Ho} = e^{\frac{-t}{T}}$$

The equalization ratio, E, is determined by

$$E = \frac{Ho - H}{Ho} = 1 - e^{\frac{-t}{T}} \qquad (7)$$

Equation (7) for the equalization ratio is valid only if the initial assumptions are satisfied, and is applicable only to conditions of constant ground-water pressures or when the changes in the ground-water pressure are instantaneous. For a hydraulic or diaphragm piezometer installation, equation (7) can be modified by using pressures instead of heads, and by replacing the area of the standpipe, A, in equation (4) by the volume factor, λ, multiplied by the unit weight of water. By means of equation (7) response times for any percentage of equalization can be calculated.

3.3 Kallstenius' Theory

Kallstenius (1956) presented a theoretical solution for estimating time lags corresponding to any degree of equalization. His theory is based on assumptions similar to those of Hvorslev's (1951), and gives nearly identical results.

It is assumed that:

1. The soil layer is saturated, isotropic and infinite in extent.
2. Water flowing to the piezometer comes from a great distance.
3. The piezometer intake may be considered as a sphere with equivalent surface area.
4. The piezometric level in the ground is constant; and, at zero time, the observed pore pressure differs from this by Δu.

Consider a piezometer installation for which at time, t, $u_s - u_i = \Delta u$ (8)

where

u_s = constant piezometric pressure in the ground, gm/cm^2

u_i= observed pore pressure at time t, gm/cm^2

Δu= error in reading, gm/cm^2

By differentiating equation (8) with respect to time,

$$\frac{\partial \Delta u}{\partial t} = \frac{-\partial u_i}{\partial t} \tag{9}$$

The flow of water through the soil to the tip at time, t, may be given by

$$V_u = \frac{k\ i\ A_r}{\gamma_w}$$

$$= \frac{k}{\gamma_w} A_r \frac{\partial \Delta u}{\partial_r} \tag{10}$$

where

V_u = rate of flow, cm^3/sec

k = permeability of soil cm/sec

i = pressure gradient gms/cm^3

r = distance from piezometer intake to a certain definite soil element, cm

A_r = surface area of a sphere with radius r, cm^2

γ_w = unit weight of water, gm/cm^3

The rate of flow into the piezometer system, if the system has a volume factor of λ cm^5/gm is,

$$V_u = \frac{\partial u_i}{\partial t} \lambda \tag{11}$$

Re-arrangement and integration of equation (10) with the boundary conditions $r_o < r < \infty$, where r_o is the radius of a sphere with a surface area, A, equivalent to that of the intake filter, yields,

$$\Delta u = \frac{V_u\ \gamma_w}{2k\sqrt{\pi A}} \tag{12}$$

Combining equations (9), (11), and (12) results in a differential equation,

$$\frac{\Delta u}{\partial t} + \frac{2k\sqrt{\pi A}}{\gamma_w \lambda} \Delta u = 0 \qquad (13)$$

Solution of equation (13) given

$$\frac{\Delta u}{(\Delta u)t=0} = e^{-\frac{2k\sqrt{\pi A}}{\gamma_w \lambda} t} \qquad (14)$$

By comparing equation (14) with equation (7), where for a hydraulic or diaphragm piezometer $T = \frac{\lambda \gamma_w}{F k}$, it can be seen that the Hvorslev and Kallstenius theories differ only in the method of accounting for the size and shape of the intake; and for a spherical intake, identical answers are obtained.

3.4 Gibson's Theory

By ignoring the compressibility of the soil skeleton, the Hvorslev and Kallstenius theories, strictly speaking, apply only to the case of time lags in coarse grained soils. Gibson (1963) recognized this limitation and developed a time lag theory which takes into account the compressibility of the soil in which measurements are being taken. His theory is based on the assumptions that:

1. Terzaghi's consolidation theory is applicable.
2. The soil is fully saturated, isotropic, and homogeneous.
3. There is no undissolved gas in the measuring system.
4. The permeability of the porous intake is much larger than that of the soil.
5. The porous element can be replaced by an equivalent spherical element of radius, a , which is very small in comparison to its depth below ground surface.

6. The initial pore pressure, u , is sensibly uniform around the porous intake, and this uniformity extends to a radius beyond which the transient phenomena of equalization hardly manifests itself.

Consider the case of an open standpipe (Fig. 3.2) with the porous intake replaced by an equivalent sphere of radius, a. Since the pore pressure in the soil is assumed initially to be uniform,

$$u = u_o \quad \text{at} \quad t=0,\ r>a \tag{15}$$

where u = pore pressure at any time at any point in the soil mass

u_o = constant pore pressure at an infinite distance away from the piezometer tip.

t = time

r = distance from the center of the tip to any point in the soil mass.

Now if it is considered that at zero time the observed pore pressure is out of balance with the ground pore pressure, and is equal to $h_{(o)}\gamma_w$, then at any time, t , after the initiation of equalization, the pore water, $u(r,t)$, is governed by the consolidation theory in spherically symetrical form,

$$c_v \left(\frac{\partial^2 u}{\partial r^2} + \frac{2}{r}\ \frac{\partial u}{\partial r}\right) = \frac{\partial u}{\partial t}\ ,\ r>a \tag{16}$$

where C_v= coefficient of consolidation.

At an appreciable distance from the intake the pore pressure will be unaffected by equalization and:

$$u \rightarrow u_o \quad \text{as } r \rightarrow \infty \tag{17}$$

The rate at which water flows out of the clay must equal the rate at which water enters the standpipe so:

$$4\pi a^2 \frac{k}{\gamma_w}\left(\frac{\partial u}{\partial r}\right)_{r=a} = A\ \frac{dh}{dt} \tag{18}$$

where k = permeability of the soil

A = cross-sectional area of the standpipe

dh = change in elevation of the water level in the standpipe.

The water pressure, at any time after the start of equalization, must be continuous between the intake and the soil, hence:

$$u(a,t) = \gamma_w \, h(t) \qquad \text{for } t > 0 \qquad (19)$$

By expressing the stiffness of the measuring system, μ , and the time factor, T, as follows,

$$\mu = \frac{4\pi a^3 m_v \gamma_w}{A} \qquad (20)$$

and

$$T = \frac{c_v t}{a^2} \qquad (21)$$

and by using the conditions expressed by equations (15) to (21), Gibson obtained a solution for the equalization ratio, ϵ*, at any time:

$$\epsilon = \frac{h(\infty)-h(t)}{h(\infty)-h(0)} = \frac{1}{n_1-n_2}\left[n_1 \exp(n_1^2 T)\operatorname{erfc}(n_1 T^{\frac{1}{2}}) - n_2 \exp(n_2^2 T)\operatorname{erfc}(n_2 T^{\frac{1}{2}})\right]$$

where

$$\left.\begin{matrix} n_1 \\ n_2 \end{matrix}\right\} = \tfrac{1}{2}\left[\mu \pm (\mu^2 - 4\mu)^{\frac{1}{2}}\right]$$

and

$$\operatorname{erfc} x = 2\pi^{-\frac{1}{2}} \int_x^\infty \exp(-\zeta^2)\, d\zeta$$

Gibson has computed numerical values from equation (22) and has plotted the results as families of curves on three separate graphs. By means of these graphs, and equations (20) and (21), it is relatively easy to calculate the time

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* With Hvorslev's theory the equalization ratio is defined as $E = \dfrac{h(t)-h(0)}{h(\infty)-h(0)}$ so $\epsilon = 1 - E$.

required for any percentage equalization.

It is of interest to note that if m_v of the soil is allowed to approach zero, as is tacitly assumed in Hvorslev's theory, Gibson's solution for the equalization ratio simplies to $\epsilon = e^{-\frac{4\pi akt}{A}}$ which corresponds exactly with the relationship developed by Hvorslev for a spherical tip. Gibson's theory is applicable for a hydraulic or diaphragm piezometer if A, in equation (18), is replaced by the volume factor multiplied by the unit weight of water.

CHAPTER IV

LABORATORY TESTS

4.1 General

On the basis of reported and anticipated satisfactory performance, one open standpipe, two hydraulic, and two diaphragm piezometers were selected for laboratory response tests. This chapter describes the piezometers tested, the test apparatus used, the properties of the soil used, and the procedures followed.

4.2 Piezometers Tested

The following piezometers were tested:

1. The Maihak MDS 75 vibrating wire, diaphragm piezometer (Fig. 2.1)
2. The Warlam air operated, diaphragm piezometer (Fig. 2.15)
3. The Bishop high air entry, hydraulic piezometer (Fig. 2.10)
4. The U.S.B.R. high air entry, foundation type, hydraulic piezometer (Fig. 2.1)
5. The Geonor open standpipe piezometer (Fig. 2.14).

Every attempt was made to connect the various piezometers to their respective pressure monitoring instruments as they would be in an actual field installation. Special measuring cable Type GG was used to connect the Maihak MDS 75 to the Maihak MDS 3 receiver. The Warlam piezometer was connected, by short lengths of 3/16" OD by 0.11" ID Nylaflow Type H 2500 psi tubing, to the Warlam monitoring instrument. Nylon tubing, 3/16" OD by 0.107" ID with a 0.040" coating of polythene, was used to attach the Bishop piezometer first to a saturated Bourdon gauge, and later to a mercury manometer. Short lengths of 5/16" OD by 1/4" ID polyvinylidene chloride

(Saran) tubing were used to connect the U.S.B.R. tip to a saturated Bourdon gauge. The standpipe of the Geonor piezometer was of 3/8" OD by 1/4" ID polyethylene tubing. Table 4.1 lists the cost as well as the name and address of the supplier of the piezometer equipment purchased for this investigation.

4.3 Apparatus

The test apparatus consisted of a steel tank 12 inches in diameter and 24 inches high, fitted with a rubber membrane, inside of which was placed remolded clay. A conical shaped opening which would accomodate a number 12 rubber stopper was provided in the lid so that the leads of the various piezometers being tested could be brought out of the tank. An air cylinder, with a sensitive pressure regulator, was connected to a water tank half full of water, which in turn was connected through a Standard Wykeham Farrance constant pressure control manifold to a mercury manometer and to the test tank (Fig. 4.1). By this arrangement increments of pressure could be applied to the membrane enclosed soil.

4.4 Soil Properties and Preparation

The soil in which the piezometers were tested was a remolded, highly plastic, glacio-lacustrine clay. Classification tests in accordance with ASTM procedures yielded the following results:

Liquid limit	72.5%
Plastic limit	32.1%
Plasticity index	40.4%
Specific gravity	2.79
Grain size,	
sand	4%
silt	35%
clay	61%

Approximately 120 pounds of air dried soil, crushed to pass a #40 sieve, was mixed at a liquidity index of 0.94, formed into a conical pile, covered with a sheet of plastic, and left in the moist room for two days to ensure uniformity of moisture content. To fill the test tank, soil was scraped from the stockpile in small quantities, using a spatula, and thrown into the tank. Control tests performed by filling a standard Proctor mold using this procedure showed that a degree of saturation between 95.9 percent and 97.5 percent could be expected.

The permeability and consolidation characteristics of the soil were determined in order to compare the response versus time curves, obtained from the various tests, with the theoretical relationships, derived by Hvorslev 1951 and Gibson 1963). The permeability of the clay at various moisture contents, Fig. 4.2, was calculated from its rate of consolidation, and was measured in a constant head permeability test. A procedure similar to that outlined by Casagrande and Fadum (1940) was used in the constant head permeability test. A constant pressure was applied, through a twin burette volume change indicator, to one side of a clay specimen in a fixed ring oedometer. At the placement moisture content of 70 percent it was found that the soil had a permeability, k, of 2.5×10^{-7} cm/sec; a modulus of volume change, m_v, of 8.3×10^{-4} cm^2/gm; and a coefficient of consolidation, c_v, of $3 \times 10^{-4} cm^2/sec$.

4.5 Procedure

In general, each saturated piezometer tip in turn was carefully pushed into the clay in the test pot to a central location, and the hole above it refilled with clay. The leads were brought through the lid, a water supply was connected to the tank and, with water overflowing to avoid trapping air, the lid was bolted down. The instrumented test tank was then

left a minimum of two days to obtain equilibrium conditions. Elevation heads between the center of the piezometer tip and the manometer and Bourdon gauges were carefully measured and the pressure monitoring instruments were zeroed, de-aired and calibrated as required.

With the valve at the test tank closed, a predetermined pressure was applied to the rest of the system using the air cylinder and pressure regulator. At the start of a test, the valve at the test tank was opened and the induced pore pressures were monitored. Readings of the pore pressure and applied pressure were taken at increasing time intervals until such time as the pore pressure readings became constant. For tests lasting more than approximately 30 minutes, the air pressure system was isolated and a hydraulic system was then used to maintain a constant applied pressure. Most of the piezometer tips were tested at four approximately equal pressure increments, between 5 and 25 psi. The following paragraphs outline specific test procedures used with each of the five piezometer tips.

The Maihak MDS 75 was received with a protective cap over the diaphragm; the oil saturated porous stone being shipped in an oil filled, air-tight container. Manufacturer's instructions were carefully followed in screwing the stone onto the tip to ensure an air free connection and to avoid over-stressing the diaphragm. With the tip acted upon only by atmospheric pressure a number of zero readings were taken. The tip was then calibrated (Fig. 4.3), using the test tank, the tank being filled with water. The applied pressure was read on a mercury manometer. Subsequently, the test tank was filled with remolded clay and the response of the piezometer was determined under applied pressure increments of approximately 5-10, 10-15, 15-20 and 20-25 psi. Periodically during the calibration and the response tests,

the frequency of the comparator wire in the receiver was checked against a tuning fork built into the receiver, and adjustments were made if necessary.

The Warlam piezometer tip was placed in the clay filled test tank, connected to its pressure monitoring apparatus, and subjected to pressure increments approximately the same as for the Maihak tip.

The Bishop piezometer tip and short lengths of connecting tubing were placed under distilled water in a vacuum desiccator to saturate the tip. Once saturated, the tip was pressed into the clay, with the ends of the connecting tubing in water so that water siphoned to the tip and dripped from the stone. One of the tubes was connected to a needle valve and the other to a Bourdon gauge. The connections were made in such a manner that no air was trapped in the lines. The 4-1/2" Marsh test gauge used was complete with a Type DBX diaphragm attachment modified so that no air was trapped below the diaphragm. Four pressure increments between 5 and 25 psi were applied to the Bishop tip.

When the tests on the Bishop tip connected to the Bourdon gauge had been completed, the gauge was disconnected and a mercury manometer was attached. Care was taken to avoid trapping air between the tip and the mercury column of the one leg of the manometer. This arrangement was tested at four pressure increments of approximately 5 psi. In a further test on the Bishop piezometer a large bubble of air was introduced between the tip and the manometer. Pressure increments of approximately 0-5 and 5-10 psi were then applied.

The U.S.B.R. foundation piezometer tip was saturated and installed in the test tank using the same procedure as for the Bishop tip. The tip was then connected to the Marsh gauge and tested at four applied pressure increments of approximately 5 psi each between 5 and 25 psi.

The Geonor tip was connected to a polyethylene standpipe and enclosed by a rubber membrane. The tip and a section of the tubing were then filled with de-aired water. As the tip was pushed into the clay, the membrane was rolled back so that the installation was accomplished with negligible entrapped air. The standpipe was brought vertically from the test tank and was secured at a point near the ceiling. A graduated scale fastened to the outside of the standpipe enabled the elevation of the water level to be measured. A pressure increment of approximately 6 feet of water was applied and readings of the water level were taken until a constant reading was obtained.

The size of the filter intake for each of the five piezometers was carefully measured and a corresponding shape factor was calculated using the equations in Table 4.2. Volume factors were determined for all but the Maihak and Warlam piezometer systems. For the Bishop manometer and the Geonor standpipe this entailed determining the cross-sectional area of the respective tubing and was accomplished by a number of measurements of the volume of water filling known lengths of the tubing. The volume factors for the Bourdon gauge connected to a 10 foot length of polythene coated nylon tubing and subsequently to a similar length of PVNC tubing were determined by measuring the movement of an air-water meniscus under applied pressure increments. The distance from the gauge to the air-water meniscus in each case was carefully chosen to equal the length of tubing connected to the Bishop and U.S.B.R. tips during the response test. By knowing the cross-sectional area of the polythene coated nylon and PVNC tubing, the movement of the meniscus was converted to volume and is plotted in Fig. 4.4 versus applied pressure. Table 4.3 lists the shape and volume factors measured for the various piezometer systems tested.

CHAPTER V

PRESENTATION AND DISCUSSION OF TEST RESULTS

5.1 General

In the following sections, the test apparatus and procedure is reviewed, the accuracy of the Maihak piezometer is discussed and difficulties experienced with the Warlam piezometer are outlined. Results of response tests are compared and the use of the response time theories is discussed.

5.2 Apparatus and Test Procedure

The test procedure used to determine the response characteristics of the piezometer systems was very similar to that used in a triaxial pore pressure reaction test. A predetermined pressure was built up in the control system and applied almost instantaneously to the membrane and enclosed soil mass. Under field conditions, other than for dynamic loading, it is unlikely that such abrupt pore pressure changes would be encountered. The laboratory test procedure is therefore considered to represent the most severe conditions to which a piezometer system would be subjected. It is felt however, that a valid comparison of the response characteristics of the various piezometer systems was obtained.

At the start of a response test, when the valve at the test tank was opened, it was found that the applied pressure decreased by as much as 0.7 psi and a time of up to two minutes was required for stabilization. It was considered that this initial variation in applied pressure would have an insignificant effect on the observed pore pressure readings and no corrections were made.

Pore pressures and applied pressures were corrected, where applicable, for the difference in elevation head between the center of the piezometer pickup and the pressure monitoring apparatus. It was found for the majority of the tests, that the maximum observed pore pressures were in the order of 0.5 psi less than the corresponding applied pressure. Since the estimated degree of saturation of the soil as placed in the test tank was 95.9 to 97.5 percent, it was not anticipated that these pressures would be found equal.

The applied pressure increments were not identical in each of the tests and therefore it was not possible to make a direct comparison of the observed pore pressure versus time relationships for the various tests. For this reason, the pore pressure readings for each applied pressure increment have been converted to percent response and are shown plotted versus time (Fig. 5.1 to Fig. 5.7). Appendix A includes a typical set of data showing the pressure readings obtained, a plot of applied and pore pressure versus time, and the calculations required to convert the pore pressure readings to percent response.

5.3 Maihak Piezometer

The following paragraphs are limited to a discussion of the observed accuracy and general behavior of the Maihak piezometer. The response characteristics will be discussed in Section 5.5.

The Maihak MDS-75 transmitter was calibrated against a mercury manometer using an MDS-3 receiver, serial number 39660. A linear calibration curve, varying from approximately 0.066 psi per scale division at a pressure of 1 psi, to 0.067 psi per scale division at a pressure of 20 psi, was obtained (Fig. 4.3). For convenience, a calibration constant of 0.067 psi per scale division could be used over the pressure range from 0 to 25 psi with a maximum associated error of 0.1 psi.

Maihak quoted a calibration constant of 0.066 psi per scale division; however, it was stated that this constant was applicable only for use with receivers having serial numbers starting 55———.

The reproducibility of the pressure readings obtained from the MDS-75 can be seen in Figure 4.3. At 15 scale divisions the range in recorded pressures is only 0.01 psi, while at 390 scale divisions the range is 0.11 psi. The above figures include any errors made in reading the mercury manometer or in adjusting the receiver. It was found that the MDS-3 receiver could be adjusted to $\pm$ 0.01 psi, i.e. $\pm$ 0.2 scale divisions, and the mercury manometer could be read to $\pm$ 0.02 psi.

The foregoing discussion indicates that the accuracy of the Maihak piezometer is greater than is required for many field installations. It must be born in mind however, that once the tip has been installed in the field, there is no way of checking its calibration and the initial curve must be relied upon for the life of the installation.

5.4 Warlam Piezometer

As mentioned in CHAPTER II, the Warlam piezometer is basically a sensitive, diaphragm type check valve, shut tight by external pore pressure, and opened up by internal air pressure (Warlam 1963). The procedure for obtaining a pore pressure reading is to increase the air pressure in one of the leads to something in excess of the external hydrostatic pressure, as indicated by the flow of air through a vent lead; and, then to reduce the air pressure until the flow of air just ceases. The observed air pressure at this point is equal to the pore pressure acting on the tip, providing the check valve closes under a small pressure differential.

The Warlam piezometer was tested over a two month

period, and initially displayed a satisfactory performance. The check valve appeared to function under a small pressure differential, giving a clear indication of when a balance between the air pressure and pore pressure had been obtained. Near the end of the test period the check valve in the tip became defective and required a very large pressure differential before a positive shut-off was obtained. With a pressure of 21 psi applied to the test tank, pore pressure readings of only 6 psi were observed. Subsequent tests with water in the test tank confirmed that the tip had become defective. The tip was disassembled; however, no foreign matter was found lodged in the check valve, nor was there any sign of moisture within the body of the tip. When it is considered that the tip was handled with extreme care and the manufacturer's operating instructions were carefully followed, the value of the Warlam piezometer for field installations is rather questionable. It must be born in mind however, that only one tip was tested and it may have been defective from the manufacturer.

5.5 Response Test Results

For clarity, the results of the tests on individual piezometer systems have been plotted on separate graphs (Fig. 5.1 to Fig. 5.7). The response characteristics of the Warlam piezometer (Fig. 5.2) were determined prior to the tip becoming defective.

A common feature noticed for most of the piezometer systems tested, was that with increasing pressures a more rapid response was obtained (Fig. 5.1 to Fig. 5.5). There are two factors which are believed to have contributed to this behavior. As mentioned previously, the soil as placed in the test tank was not fully saturated. It is quite likely that, as the pressure applied to the soil was increased, some of the air in the voids was driven into solution. This would

tend to increase the degree of saturation and in turn result in a faster response. The volume factor of the Bourdon gauge, plus connecting tubing (Fig. 4.4) used for the tests on the Bishop and U.S.B.R. piezometers (Fig. 5.3 and Fig. 5.6), was found to decrease with increasing pressures. The decrease in volume factor at 25 psi, from that at 5 psi, is of sufficient magnitude to reduce the response times of the two piezometer systems by approximately 50 percent. Since the volume factors of the Maihak and Warlam piezometers were not measured, it is not known if they too tended to decrease with increasing pressure. The volume factor of the manometer used in a series of tests on the Bishop piezometer was a constant and depended only on the internal diameter of the manometer tubing. The results of tests on this piezometer system, (Fig. 5.4), show less of a tendency for a decrease in response time with increasing pressures.

For the purpose of comparison, the response curve for the lowest pressure increment was selected from each of Figures 5.1 to 5.7 and replotted on Figure 5.8. A large variation in the time required for 100 percent response was observed for the different piezometer systems, from one minute for the Maihak tip to approximately 4000 minutes for the Geonor open standpipe piezometer. It was previously noted that the response of a piezometer system is a function of the shape factor of the filter intake, the volume factor of the system, and the properties of the soil in which the tip is installed. Since the soil properties were not changed during the response tests, the typical response characteristics shown on Figure 5.8 are affected only by the differences in volume and shape factors of the various piezometer systems (Table 4.3).

For the response tests on the Bishop piezometers, the tip was connected in turn to a Bourdon gauge, a mercury

manometer, and a mercury manometer with air introduced between the tip and the manometer. The corresponding volume factors were, 1.47, 11.3, and 18.6 x 10^{-4} cm^5/gm.; and, the observed times required for 100 percent response were 2.5, 30, and approximately 300 minutes, respectively. It should be pointed out that in field installations the volume factor of a hydraulic piezometer system is greatly affected by the length and flexibility of the connecting tubing. Under such conditions, the response characteristics of the piezometer system may not be significantly affected whether a Bourdon gauge or a manometer is used to measure the pore pressure.

The effect of different shape factors can be seen by comparing the response curves for the Bishop and U.S.B.R. piezometers, both connected to the same Bourdon gauge. The Bishop piezometer tip has a fairly large filter intake, and a shape factor of 40.9 cm.; whereas, the U.S.B.R. tip has a small intake, and a shape factor of only 5.3 cm. Observed times for full response were 2.5 minutes for the Bishop, and 80 minutes for the U.S.B.R. piezometer.

Volume factors were not determined for the Maihak and Warlam diaphragm type piezometers. The difference in shape factors, 26.6 cm. for the Maihak and 6.7 cm. for the Warlam, would however, account in part for the dissimilar response characteristics observed. Pore pressure equalization required one minute for the Maihak, compared to 10 minutes for the Warlam piezometer.

The effect of the volume factor on the response of an instrument is most clearly illustrated by the response times observed for the Geonor open standpipe piezometer. Although this piezometer had a large shape factor, 61.1 cm., it also had a very large volume factor, 0.266 cm^5/gm. Approximately 4000 minutes were required for the equalization of pore pressures. For many field

installations, the piezometer tip is located in a sand filter. The shape factor of a sand filter, 5 feet long by 4 inches in diameter, was calculated to be 280 cm.. By use of such a filter, the response times for the Geonor piezometer could have theoretically been reduced by a factor of 4.5. Even with such a reduction, the response times for the Geonor piezometer system would have been longer than for any of the other systems tested.

It must be considered that the response relationships presented in this chapter were determined using a remolded clay with a permeability of 2.5×10^{-7} cm/sec. For an insitu clay, and in particular an intact overconsolidated clay, much lower permeabilities and proportionately longer response times could be expected. Hence, although it is only one of the factors influencing the satisfactory performance of a piezometer system, for low permeability soils the rate of response assumes increasing importance.

5.6 Theoretical Response Curves

With the exception of the Maihak and the Warlam tips, Hvorslev's and Gibson's theories have been used to calculate theoretical response curves for the piezometer systems tested. These theoretical response curves are included in Figures 5.3 to 5.7. The graphical solutions given for Gibson's equalization equation did not permit the calculation of response times for greater than 99 percent equalization.

For the lower percentages of equalization, the two theories gave widely differing answers; the Gibson theory indicating much faster response times. At 99 percent response, much better agreement was observed; in some cases identical response times were indicated. Since the two theories are based on different assumptions (CHAPTER III), it was not expected that they should give identical results.

The theory proposed by Hvorslev assumes the soil is incompressible. Gibson on the other hand, considers a more general case where the consolidation characteristics are taken into account. Therefore, it was anticipated that of the two theories, Gibson's would more closely approximate the laboratory test results. As can be seen this is not the case. In general, good agreement was found, at all percentages of equalization, between the observed response curves and the curves calculated by Hvorslev's theory. It is interesting to note that Penman (1961), in similar type tests, found that the observed response at lower percentages of equalization, was, in general, considerably faster than that calculated using Hvorslev's theory. It would appear that, had his results been analysed with respect to Gibson's theory, much closer agreement would have been obtained. It is believed that the difference between the results of Penman and those of this investigation may be attributed to slightly different test apparatus, test procedures, and soil characteristics.

The results of this investigation, as well as those of other laboratory and field tests (CHAPTER II), indicate that Hvorslev's theory can be used to provide a reasonable estimate of the time required for any piezometer system to indicate 95 percent equalization of the pore pressures. If this is the case, the theory could be used to advantage in indicating the relative responsiveness of various piezometer systems; and, thus permit a preliminary selection of the type best suited for specific conditions and purposes.

The time required for 95 percent response is shown as a function of soil permeability in Figure 5.9. This figure illustrates a valuable use of the response theories. The graph includes six types of piezometer systems as follows:

1. A diaphragm piezometer with characteristics given by Penman (1961).

2. A Bishop hydraulic piezometer connected to a Bourdon gauge with short leads.
3. A Bishop piezometer connected to the same Bourdon gauge by 980 feet of polythene tubing.
4. A Geonor open standpipe piezometer with 0.25 inch ID tubing, installed in a sand filter 4 inches in diameter and 5 feet long.
5. A Geonor piezometer without a sand filter.
6. A 2 inch ID open standpipe with a sand filter 4 inches in diameter and 5 feet long.

The relationships shown were calculated from the equation

$$t_{95} = \frac{3.0 \; \lambda \; \gamma_w}{F \quad k}$$ (Hvorslev 1951).

If the maximum acceptable time for 95 percent response is 3 days, then according to Figure 5.9 an open standpipe piezometer is restricted to use in soils with a permeability greater than 10^{-8} cm/sec. Depending on the length and flexibility of connecting tubing, hydraulic piezometers if properly de-aired have satisfactory response characteristics in soils with a permeability as low as 10^{-10}cm/sec. The limitations of this latter piezometer type have been discussed previously. For deep foundation investigations, if the soil has a permeability less than 10^{-8} cm/sec., the choice of piezometer is limited to a diaphragm type.

It is suggested that Figure 5.9 provides a valuable means of selecting a piezometer system compatible with given field conditions.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 General

The purpose of this study was to review the comparative aspects of various piezometer designs. To accomplish this, an extensive literature review was conducted, and five selected types of piezometers were tested in the laboratory for response characteristics. The conclusions and recommendations for further research, arising from this investigation, are presented in this chapter.

6.2 Conclusions

The conclusions which may be drawn as a result of this investigation are as follows:

1. The response time of a piezometer installation, although only one of the factors influencing the satisfactory measurement of field pore pressures, assumes increasing importance as the permeability of the soil in which the installation is to be made decreases.

2. Hvorslev's theory (CHAPTER III) can be used to advantage in calculating the relative response characteristics of various piezometer systems, thus permitting the preliminary selection of a type most suited for specific conditions and purposes.

3. Open standpipe piezometers are simple and rugged in design, but may be expected to have excessively long response times in highly impermeable soil.

4. Hydraulic type piezometer systems have a long history of satisfactory performance, however, they are limited to locations where the leads do not have to rise more than approximately twenty feet above the lowest piezometric level.

Their satisfactory performance depends to a large extent on the properties of the tubing selected to connect the piezometer tip to the Bourdon gauge or manometer. The response characteristics of a hydraulic piezometer are largely a function of the length and flexibility of the connecting tubing.

5. Diaphragm type piezometers, in general, have a very low volume factor and a correspondingly fast response; and, from this point of view, are ideally suited for pore pressure measurements in low permeability soils. As yet, however, there are few cases, if any, where the reliability and accuracy of field installations of such instruments, over a long period of time, have been established beyond doubt.

6. Recent modifications in the design of the Maihak piezometer should eliminate a number of the difficulties experienced with this tip by Moore (1960) in the Eucumbene dam. On the basis of the available evidence, of the various diaphragm type piezometers available, equal or superior field performance might be expected from the Maihak piezometer tip.

7. The Warlam piezometer tip became defective after two months of testing, and on the basis of this limited experience, might be considered of questionable value.

6.3 Recommendations

As a result of this investigation, it is recommended that further research into the comparative aspects of piezometer designs, be conducted under field conditions. This research should be directed towards establishing the long term reliability of diaphragm type piezometers, and towards determining the magnitude of the errrors associated with the slow response characteristics of some piezometer designs. It is visualized that the various piezometer designs might be installed in a saturated clay deposit which is to be subjected to a well defined surface loading. The measured

change in pore pressure, resulting from a change in stress regime, could then be compared with a theoretical value calculated from the results of triaxial tests. A research program similar to this, but using only one type of piezometer, was conducted by Gibson (1961).

LIST OF REFERENCES

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Bazett, D.J., "Field Measurement of Pore Water Pressures", Proceedings of the Twelfth Canadian Soil Mechanics Conference, Technical Memorandum No. 59, Associate Committee on Soil and Snow Mechanics, National Research Council, Ottawa, 1959.

Bazett, D.J., Discussion, Proceedings of the Soil Conference on Pore Pressures and Suction in Soils, Butterworths, London, p. 134, 1961.

Biemond, C., "Direct Measuring of Internal Water Pressures In Clay", Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1936.

Bishop, A.W., Kennard, M.F., and Penman, A.D.M., "Pore Pressure Observations at Selset Dam", Proceedings of the Conference on Pore Pressure and Suction in Soils, Butterworths, London, 1961.

Bjerrum, L., and Johannessen, I., "Pore Pressures Resulting From Driving Piles in Soft Clay", Proceedings of the Conference on Pore Pressure and Suction in Soils, Butterworths, London, 1961.

Boiten, R.G., and Plantema, G., "An Electrically Operating Pore Water Pressure Cell", Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1948.

Bozozuk, M., "Description and Installation of Piezometers for Measuring Pore-Water Pressures in Clay Soils", Building Note No. 37, Div. of Building Research, National Research Council, Ottawa, 1960.

Brooker, E.W., Personal communications with Dr. Brooker, Associate Professor of Civil Engineering, University of Alberta.

Casagrande, A., "Non-Metallic Piezometer for Measuring Pore Pressure in Clay", Appendix to "Soil Mechanics in Design and Construction of Logan Airport", Boston Society of Civil Engineers, Vol. 36, No. 2, 1949.

Cooling, L.E., Discussion, Proceedings of the Conference on Pore Pressure and Suction in Soils, Butterworths, London, p. 121, 1961.

Daehn, W.W., "Development and Installation of Piezometers for the Measurement of Pore-Fluid Pressures in Earth Dams", American Society for Testing Materials, Preprint, 1962.

de Lucia, E.R., "Earth Dam Construction in a Wet Climate", Sixth Congress on Large Dams, 1958.

Eden, W.J., "Field Studies on the Consolidation Properties of Leda Clay", Proceedings of the Fourteenth Canadian Soil Mechanics Conference, Technical Memorandum No. 69 Associate Committee on Snow and Soil Mechanics, National Research Council, Ottawa, 1961.

E.N.R., Engineering News Record, Vol. 172, No. 7, 1964.

Gibson, R.E., and Marsland, A., "Pore Water Pressure Observations in a Saturated Alluvial Deposit Beneath a Loaded Oil Tank", Proceedings of the Conference in Pore Pressure and Suction in Soils, Butterworths, London, 1961.

Gibson, R.E., "An analysis of System Flexibility and its Effect on Time-Lag in Pore-Water Pressure Measurements", Geotechnique, Vol. 13, No. 1, 1963.

Goldbeck, A.T., "The Measurement of Earth Pressures on Retaining Walls", Proceedings of the Eighteenth Annual Meeting of the Highway Research Board, 1938.

Gould, J.P., "Analysis of Pore Pressure and Settlement Observations at Logan International Airport", Harvard Soil Mechanics Series, No. 34, 1949.

Hardy, R.M., "Engineering Problems Involving Preconsolidated Clay Shales", Transactions, Engineering Institute of Canada, 1957.

Hardy, R.M., Brooker, E.W., and Curtis, W.E., "Landslides in Overconsolidated Clays", The Engineering Journal, The Engineering Institute of Canada, 1962.

Hardy, R.M., "The Peace River Highway Bridge - A Failure in Soft Shales", Highway Research Record, No. 17, 1963.

Hvorslev, M.J., "Time Lag and Soil Permeability in Ground Water Observations", Bulletin No. 36, Waterways Experiment Station, U.S. Corps of Engineers, Vicksburg, Mississippi, 1951.

Jones, O.T., Discussion on "Pore Pressures in Eucumbene Dam", Third Australia-New Zealand Conference in Soil Mechanics and Foundation Engineering, 1960.

Kallstenius, T., and Wallgren, A., "Pore Pressure Measurement in Field Investigations", Royal Swedish Geotechnical Institute Proceedings, No. 13, 1956.

Kallstenius, T., Discussion, Fourth International Conference On Soil Mechanics and Foundation Engineering, p. 131, 1957.

Karol, R.H., "Use of AM-9 Chemical Grout for Sealing Piezometers", Soils Engineering Research Centre, Princeton, New Jersey, 1960.

Lambe, T.W., "Sealing the Casagrande Piezometer", Civil Engineering, New York, Vol. 29, No. 4, 1959.

Lambe, T W., "Directions for the Installation of the Modified Casagrande Piezometer", Chemical Grout Technical Data, Cyanamid of Canada Ltd., 1960.

Le Moigne, G., "La Détermination des Pressions Interstitielles dans les Barages en Terre", L'Institut Technique du Batiment et des Travaux Publics, 1958.

Little, A., and Vail, A.J., "Some Developments in the Measurement of Pore Pressure", Proceedings of the Conference on Pore Pressure and Suction in Soils, Butterworths, London, 1961.

Maihak, Correspondence with J.O.W. Lence Ltd., agents for the Maihak piezometer. 1965.

McConnell, A.D., Discussion on "Pore Pressures in Eucumbene Dam", Third Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, 1960.

Moore, P.J., "Pressures in Eucumbene Dam", Third Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, 1960.

Muhs, H., "Measurement of Pore Water Pressure in Open Ground Particularly in Earth Dams", Translation from Bau-Maschine and Bautechnik, Vol. 7 and 8, 1954.

Naylor, D., Personal communication with D. Naylor, Instrumentation Engineer for the Portage Mountain Dam, British Columbia, 1965.

Painter, W.T., "An Investigation of the Le Sueur Landslide, Edmonton, Alberta", M.Sc.Thesis, Department of Civil Engineering, University of Alberta, 1965.

Penman, A.D.M., "A Field Piezometer Apparatus", Geotechnique, Vol. VI, No. 2, 1956.

Penman, A.D.M., Correspondence, Geotechnique, Vol. VIII, No. 3, 1958.

Penman, A.D.M., "A Study of the Response Time of Various Types of Piezometers", Proceedings of the Conference on Pore Pressures and Suction in Soils, Butterworths, London, 1961.

Peters, N., "Test Apparatus in Earth Embankments", Proceedings of the Twelfth Canadian Soil Mechanics Conference, Technical Memorandum No. 59, Associate Committee on Soil and Snow Mechanics, National Research Council, Ottawa, 1959.

Plantema, G., "Electrical Pore Water Pressure Cells: Some Designs and Experiences", Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1953.

P.F.R.A. Personal communication, 1964.

Ringeling, "Measuring Ground Water Pressures in a Layer of Peat, Caused by an Imposed Load", Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1936.

Robertshaw, J., Letter from J. Robertshaw, Soil Instruments Limited, to E.W. Brooker, Department of Civil Engineering, University of Alberta, 1964.

Skempton, A.W., and Henkel, D.J., "Field Observations on Pore Pressures in London Clay", Proceedings of the Conference on Pore Pressures and Suction in Soils, Butterworths, London, 1961.

Skempton, A.W., "Long-Term Stability of Clay Slopes", Geotechnique, Vol. XIV, No. 2, 1964.

Soderman, L.G., Discussion on "Field Studies on the Consolidation Properties of Leda Clay", Proceedings of the Fourteenth Canadian Soil Mechanics Conference, Technical Memorandum No. 69, Associate Committee on Soil and Snow Mechanics, Ottawa, 1961.

Speedie, M.G., "Experience Gained in the Measurement of Pore Pressures in a Dam and its Foundation", Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1948.

Speedie, M G., Discussion on "Pore Pressure in Eucumbene Dam", Third Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, 1960.

Terzaghi, K., "Measurement of Pore-Water Pressure in a Consolidating Clay Stratum", Proceedings of the Sixth Texas Conference on Soil Mechanics and Foundations Engineering, 1943.

Terzaghi, K., "Final Report of the Performance of the Ore Yard of the R.F.C. Plancor 257", From Theory to Practise in Soil Mechanics, John Wiley and Sons, New York, 1960.

Trieber, F., "Compaction Methods Adapted for the Construction of Rosshaupton Dam, Their Effectiveness and the Behavior of the Impervious Loam Core", Sixth Congress on Large Dams, 1958.

Vaughan, P., Correspondence between P.R. Vaughan, Lecturer at Imperial College, London University, England, and W.T. Painter, Graduate Student, Department of Civil Engineering, University of Alberta, 1964.

Vourinen, J., Discussion, Fourth International Conference on Soil Mechanics and Foundation Engineering, p. 129, 1957.

Vourinen, J., Discussion, Proceedings of the Conference on Pore Pressure and Suction in Soils, Butterworths, London, p. 124, 1961.

Warlam, A.A., Communciation between A.A. Warlam and E,W. Brooker, Department of Civil Engineering, University of Alberta, 1964.

Weber, W.G., "Measurement of Excess Hydrostatic Pressures in Soils", American Society for Testing Materials, Special Technical Publication No. 254, 1959.

APPENDIX A

TYPICAL EXPERIMENTAL DATA

UNIVERSITY OF ALBERTA
DEPT. OF CIVIL ENG.
SOIL MECHANICS LAB.
PIEZOMETER TEST

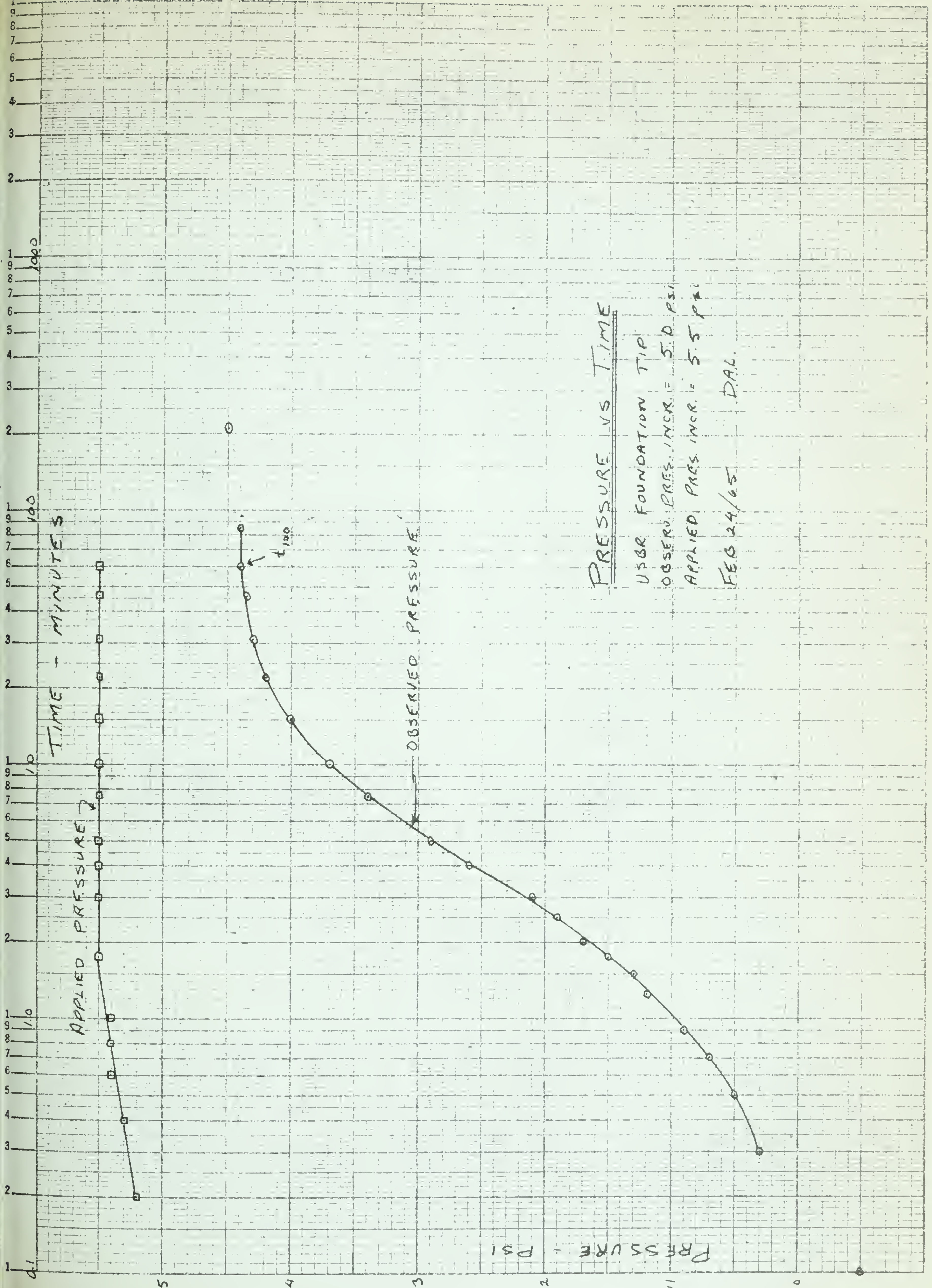
PROJECT THESIS
TYPE OF TIP USBR (FOUNDATION TYPE)
SERIAL NO. —
SOIL
ENGINEER DAL DATE FEB 24/65

~~S.D. = Scale divisions at atmospheric pressure - MDS-3 reading = 466.0 - MDS-3 reading~~

Manometer reading, psi. = Manometer reading, cm. of mercury x 0.3707

Applied pressure at tip = Manometer reading, psi. + 0.98 psi.

Time	~~MDS-3 Rdg. Scale Divs.~~	(1) ~~AS.D.~~ Bourdon Gauge psi	(2) ~~Calib.~~ Corrected ~~Const.~~ Gauge psi. ~~/S.D.~~	(3) Pore Pres. psi.	(4) Manom. Rdg. cm. of merc.	(5) Manom. Rdg. psi.	(6) Press. At Tip. psi.	Remarks
								(a) Press. Incr. = 10 to
0				9.4			10.0	15 psi
6s		6.5	8.2	9.5				(b) Bourdon gauge to
12s					38.2	14.2	15.2	tip = 37" = 1.3 psi
18s		7.2	9.0	10.3				(c) Manom. zero to
24s					38.7	14.3	15.3	tip = 26.2" ≈ 1.0 psi
30s		7.4	9.2	10.5				
36s					38.8	14.4	15.4	Note:
42s		7.6	9.4	10.7				(1) Observed pore
48s					38.9	14.4	15.4	pressure.
54s		7.8	9.6	10.9				(2) From gauge
1m					38.9	14.4	15.4	calibration curve.
1m-15s		8.1	9.9	11.2				(3) Pore pres. at
1-30		8.2	10.0	11.3				tip ie (2) + 1.3 psi
1-45		8.4-	10.2	11.5	39.0	14.5	15.5	(4) (5) Applied pres.
2m		8.6	10.4	11.7	39.0	"	"	(6) Applied press.
2-30		8.8	10.6	11.9	39.0	"	"	at tip ie
3m		9.0	10.8	12.1	39.0	"	"	(5) + 1.0 psi
4		9.4	11.3	12.6	39.0	"	"	
5		9.7	11.6	12.9	39.0	"	"	
7m-30s		10.2	12.1	13.4	39.0	"	"	
10		10.5	12.4	13.7	39.0	"	"	
15		10.8	12.7	14.0	39.0	"	"	
22		11.0	12.9	14.2	39.0	"	"	
31		11.1-	13.0	14.3	39.0	"	"	
46		11.1+	13.0+	14.3+	39.0	"	"	
60		11.2	13.1+	14.4+	39.0	"	"	← t_{100}
85		11.2	13.1+	14.4+	39.1	"	"	
210		11.2+	13.2	14.5	39.0	"	"	


PRESSURE VS TIME
USBR FOUNDATION TIP
OBSERV. PRES. INCR. = 5.0 psi
APPLIED PRES. INCR. = 5.5 psi
FEB 24/65 DAL.
TIME - MINUTES
APPLIED PRESSURE
OBSERVED PRESSURE
t100
PRESSURE - PSI
0.1
1.0
10
100
1000
15
14
13
12
11
10

CALCULATION OF PERCENT RESPONSE

USBR FOUNDATION PIEZOMETER

TIME MIN.	PORE PRES. psi	P psi	$P_o - P$ psi	$\frac{P_o - P}{P_o}$ %
0	9.4	—	—	—
0.1	9.5	4.9	0.1	2.0
0.3	10.3	4.1	0.9	18.0
0.5	10.5	3.9	1.1	22.
0.7	10.7	3.7	1.3	26
0.9	10.9	3.5	1.5	30
1.25	11.2	3.2	1.8	36
1.50	11.3	3.1	1.9	38
1.75	11.5	2.9	2.1	42
2	11.7	2.7	2.3	46
2.5	11.9	2.5	2.5	50
3	12.1	2.3	2.7	54
4	12.6	1.8	3.2	64
5	12.9	1.5	3.5	70
7.5	13.4	1.0	4.0	80
10	13.7	0.7	4.3	86
15	14.0	0.4	4.6	92
22	14.2	0.2	4.8	96
31	14.3	0.1	4.9	98
46	14.3+	0.1 -	4.9+	98+
60	14.4	0	5.0	100
85	14.4	0	5.0	100

NOTE: P_o = INITIAL OUT OF BALANCE PORE PRESSURE = $14.\cancel{5}^{4}$ − 9.4 = 5.0 PSI

P = OUT OF BALANCE PORE PRESSURE AT ANY TIME AFTER THE START OF A TEST.

$\frac{P_o - P}{P_o}$ = PERCENT RESPONSE.

TABLES

TABLE 4.1

COST AND SUPPLIER OF PIEZOMETER EQUIPMENT

PIEZOMETER SYSTEM	COMPONENTS	COST (Canadian Dollars) (Freight Not Included)	SUPPLIER
MAIHAK	a) MDS-75 Piezometer tip	$226.00	J.O.W. Lence Ltd. P.O. Box 774 Montreal 3 Quebec, Canada
	b) Type GG Conductor Cable	46.00/100 ft	
	c) MDS-3 Monitor	Approx. $3500.00	
WARLAM	a) Piezometer Tip	$186.00	Apparatus Specialties 209 Villard Avenue Hastings on Hudson New York, U.S.A.
	b) Warlam Monitor Equipment	429.00	
	c) 3/16"OD x 0.11"ID Nylaflow tubing	7.10/100 ft.	C.M. Lovested & CO. (Canada) Ltd. P.O. Box 4185 Postal Station D Vancouver 9, B.C.
BISHOP	a) Piezometer Tip	$30.00	Wykeham Farrance Engineering Ltd. 127 Edinburgh Ave. Trading Estate Slough, Bucks, England
	b) Nylon tubing 0.107" ID by 3/16"DD with 0.040" thick covering of polythene	3.70/100 ft.	Soil Instruments Ltd 45 Buck Lane London N.W. 9 England

TABLE 4.1 - Continued

U.S.B.R.	a) Polypropylene Rod	0.70/tip	Colonial Kolonite Co. 2232 W. Armitage Chicago, Illinois
	b) Ceramic Stones	$102.00/25 stones	Coors Porcelain Co. 600 - 9th Street Golden, Colorado
	c) Machining and Miscellaneous	$18.00/tip	
	d) 5/16" x 3/16" PVNC tubing	$17.71/100 ft	Industrial Plastics Canada Ltd. P.O. Box 93 Fort Erie Ontario, Canada
GEONOR	a) Piezometer tip	$32.60	Geonor Grini Molle P.O. Box 99 Roa, Oslo 7, Norway
	b) 3/8" x 1/4" Polyehtylene tubing		

TABLE 4.2

THEORETICAL SHAPE FACTORS (AFTER HVORSLEV 1951)

Shape of Filter Tip	Shape Factor F
Sphere: diameter d	$2 \pi d$
Flat Disc: diameter d	2.75 d
Cylinder: diameter d, length l	$\dfrac{2 \pi d}{2.3 \log_{10}\left[\frac{\ell}{d} + \sqrt{1+(\frac{\ell}{d})^2}\right]}$

TABLE 4.3

VOLUME AND SHAPE FACTORS OF PIEZOMETERS TESTED

Piezometer System	Volume Factor cm^5/gm.	Shape Factor cm.
Maihak Piezometer	-------------	26.6
Warlam Piezometer	--------------	6.7
Bishop Piezometer with Bourdon Gauge	*1.47 c 10^{-4}	40.9
Bishop Piezometer with Manometer	1.13 x 10^{-3}	40.9
Bishop Piezometer with Manometer (3 cc. of air in system)	1.88 x 10^{-3}	40.9
U.S.B.R. Foundation Piezometer with Bourdon Gauge	*2.1 x 10^{-4}	5.3
Geonor Piezometer	2.66 x 10^{-1}	61.1

*Average Value for 5-10 psi Pressure Increment, see Figure 4.4

FIGURES

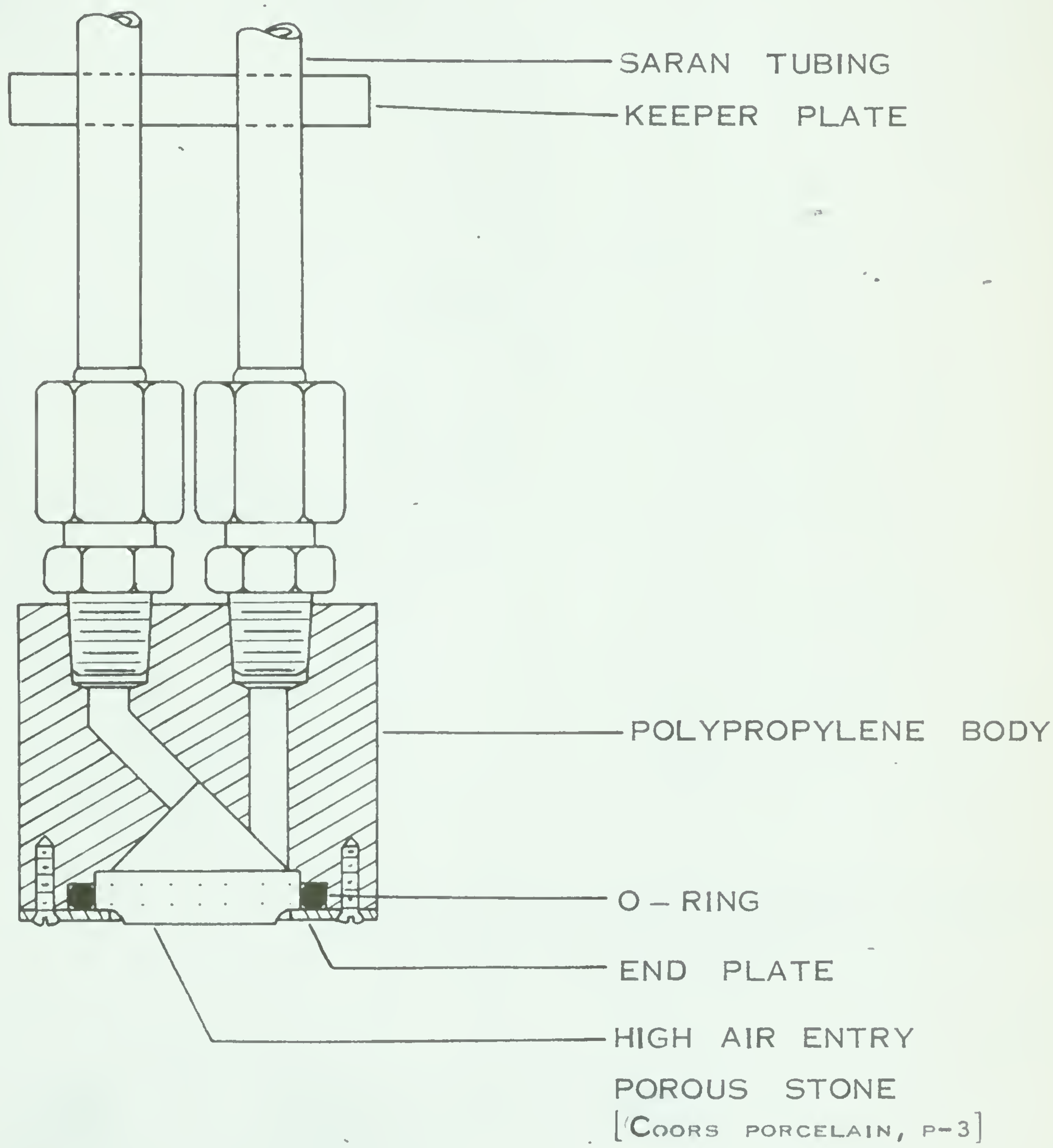


FIGURE 2.1 U.S.B.R. FOUNDATION – TYPE PIEZOMETER TIP

[AFTER U.S.B.R. DRAWING NUMBER 40 – D – 5819]

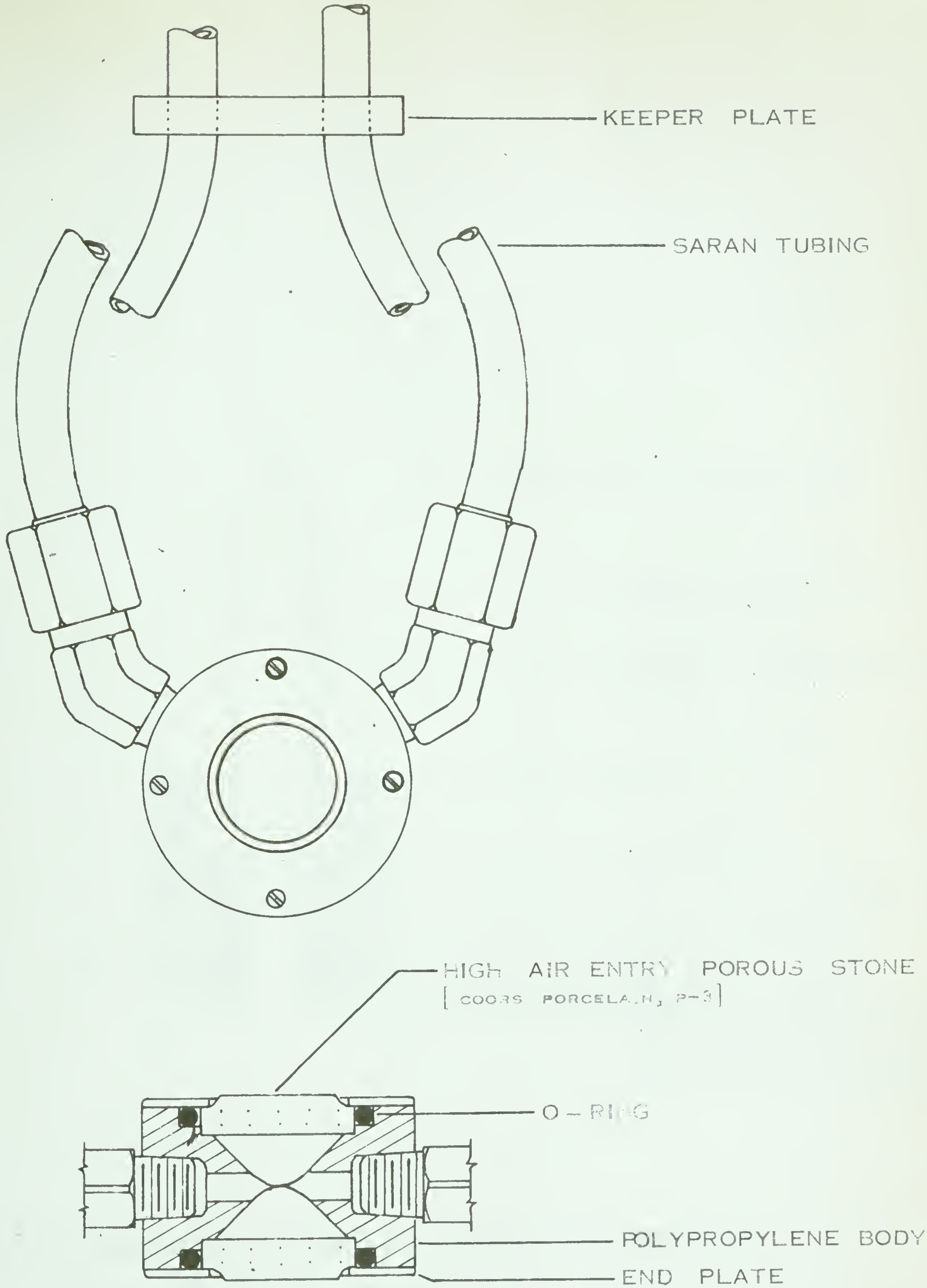


FIGURE 2.2 U. S. B. R. EMBANKMENT-TYPE PIEZOMETER TIP

[AFTER U. S. B. R. DRAWING NO. 40-D-5922

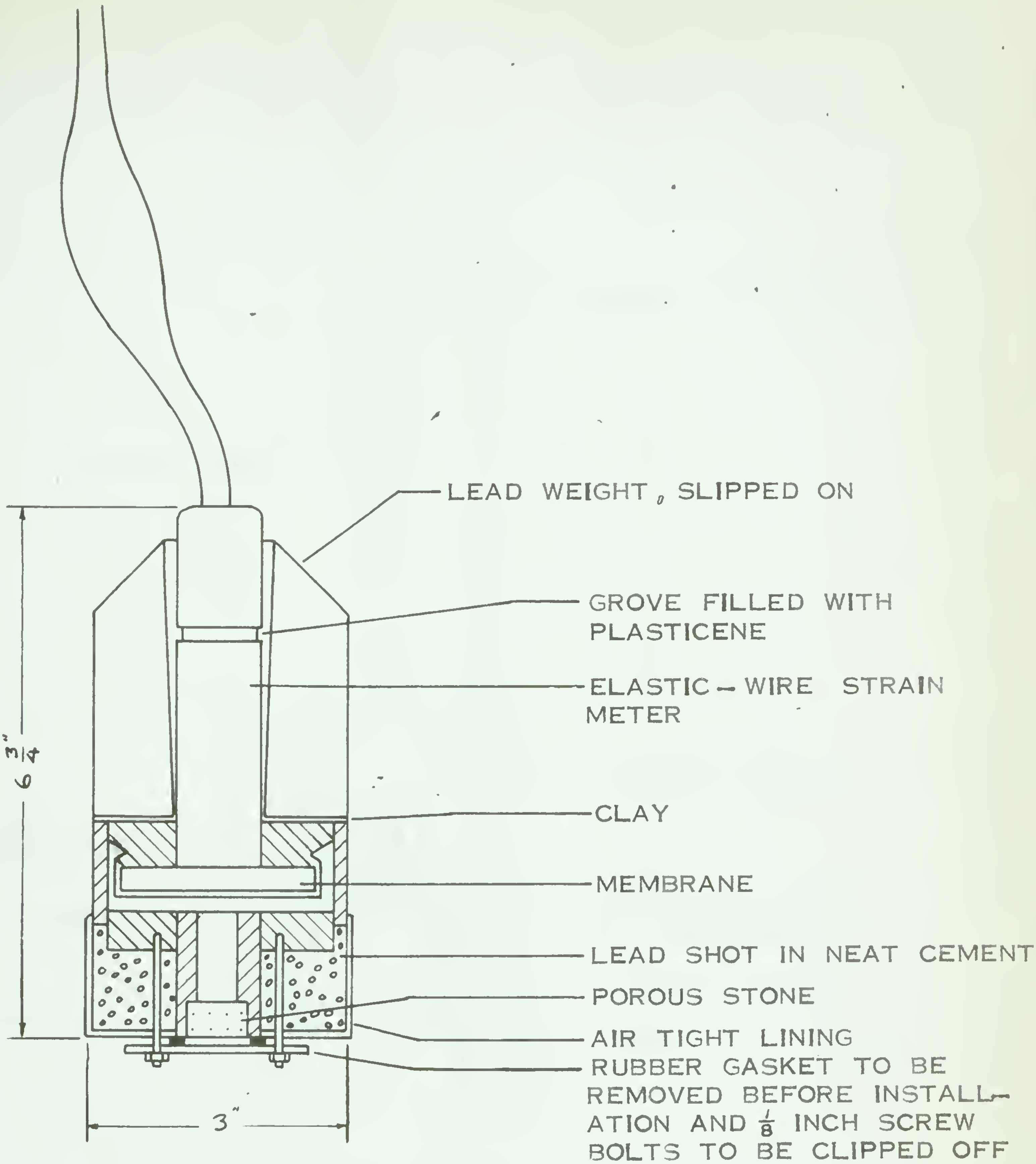


FIGURE 2.3 TERZAGHI DIAPHRAGM PIEZOMETER DESIGN

[AFTER TERZAGHI 1943]

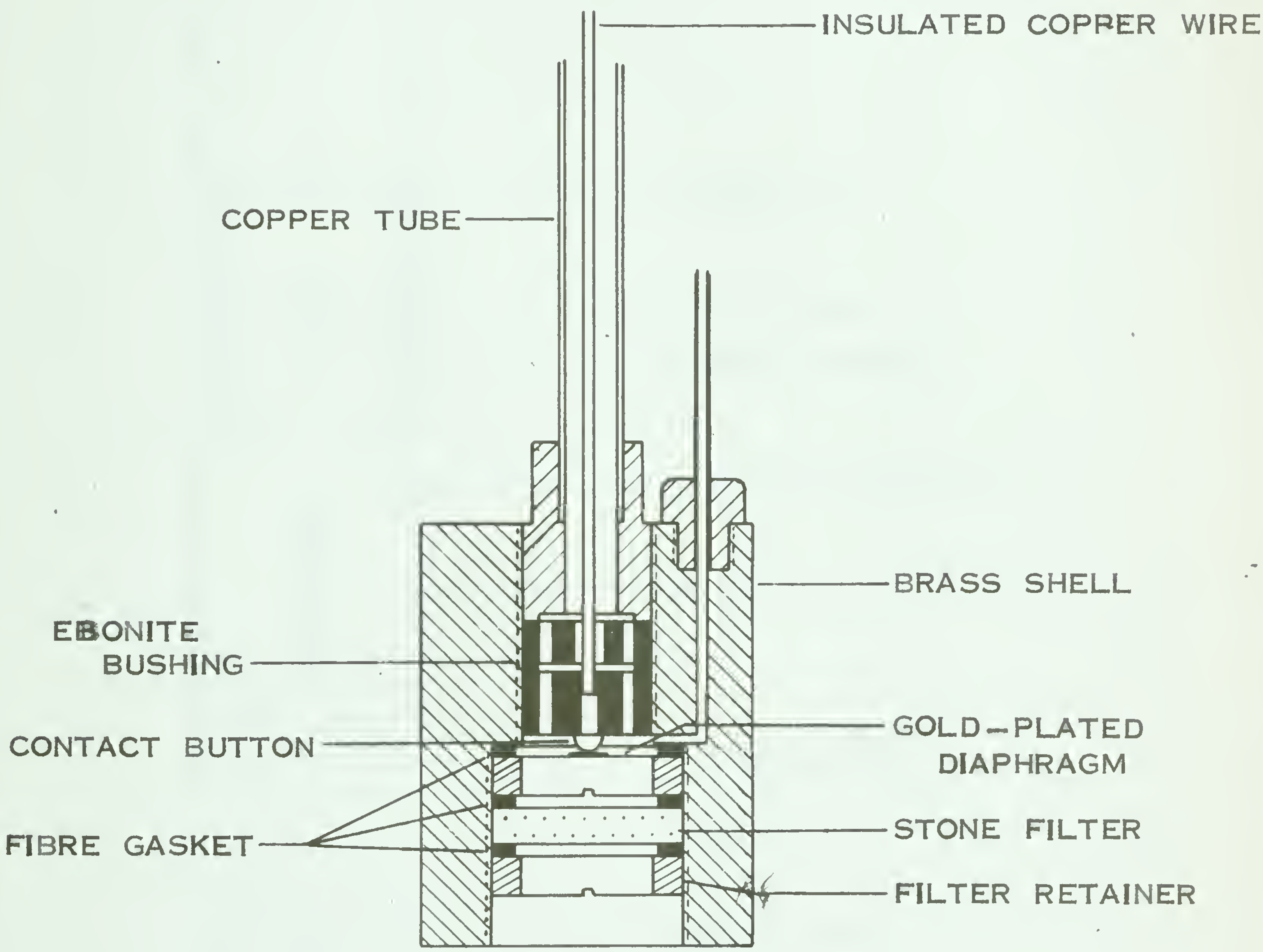


FIGURE 2.4 MODIFIED GOLDBECK PIEZOMETER [AFTER SPEEDIE 1948]

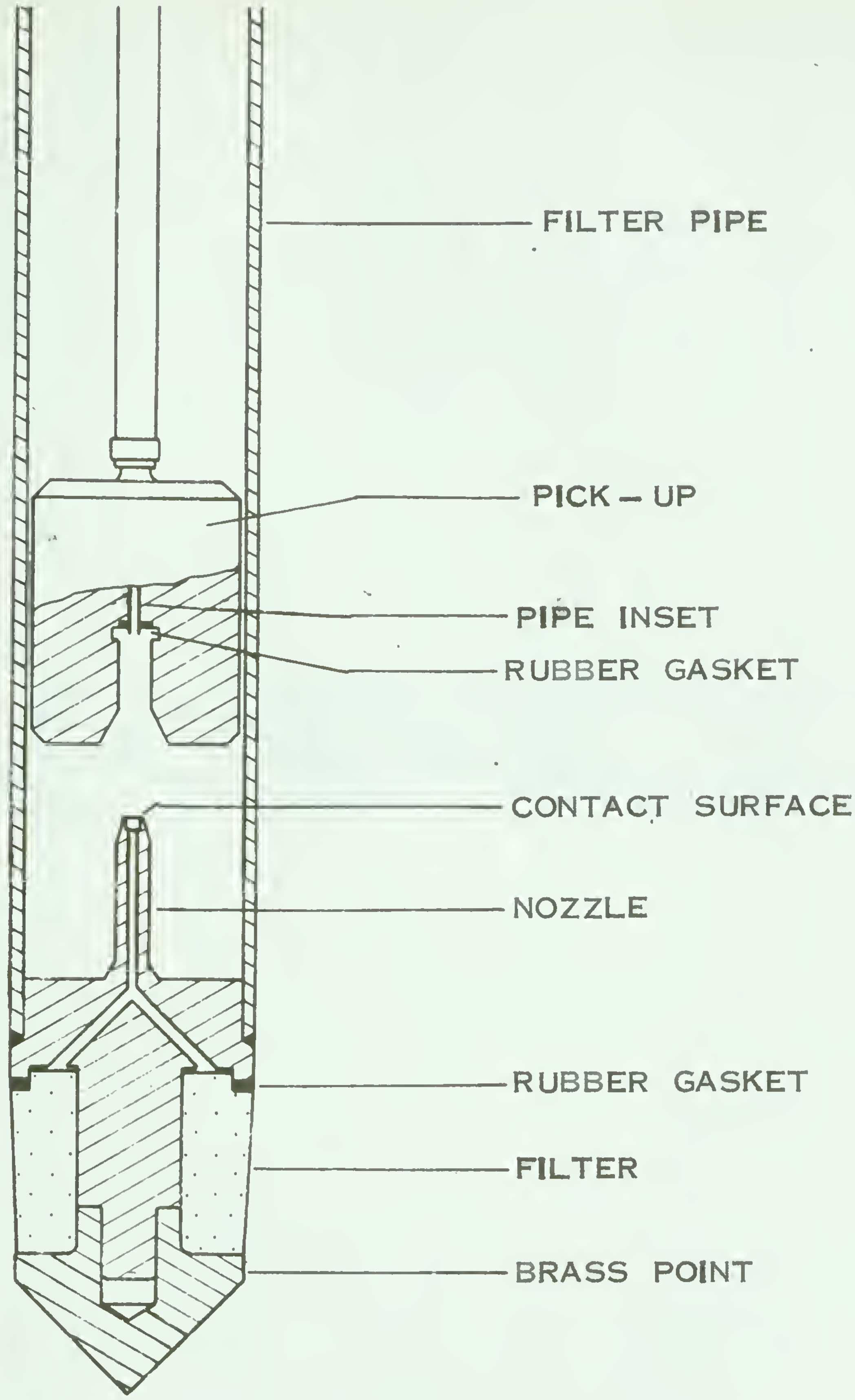


FIGURE 2.5 S. G. I. FILTER PIPE, ILLUSTRATING SPECIAL INSTRUMENT CONNECTION [AFTER KALLSTENIUS 1956]

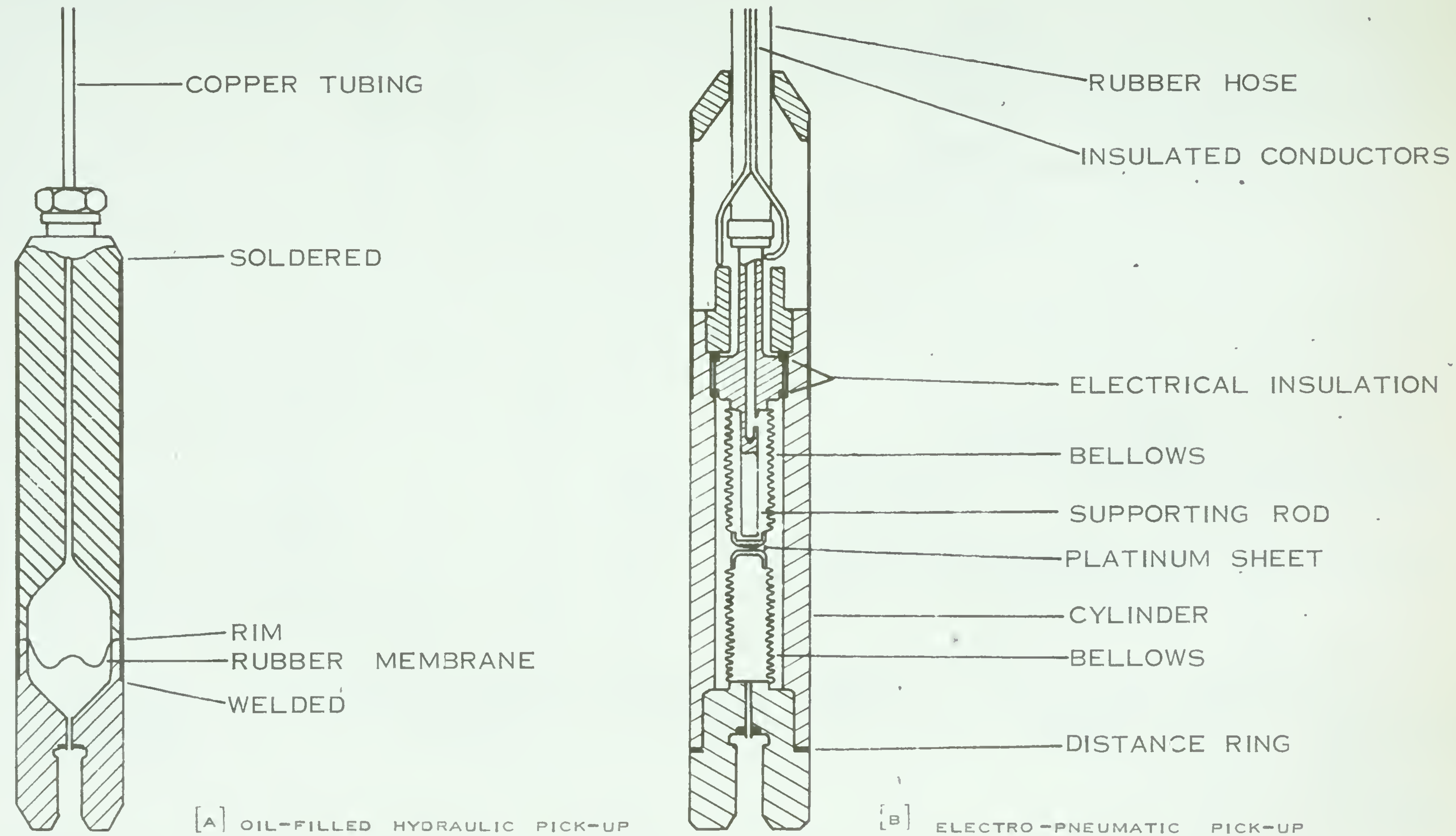


FIGURE 2.6 S.G.I. PICK-UPS USED WITH FILTER PIPE, FIGURE 2.5
[AFTER KALLSTENIUS 1956]

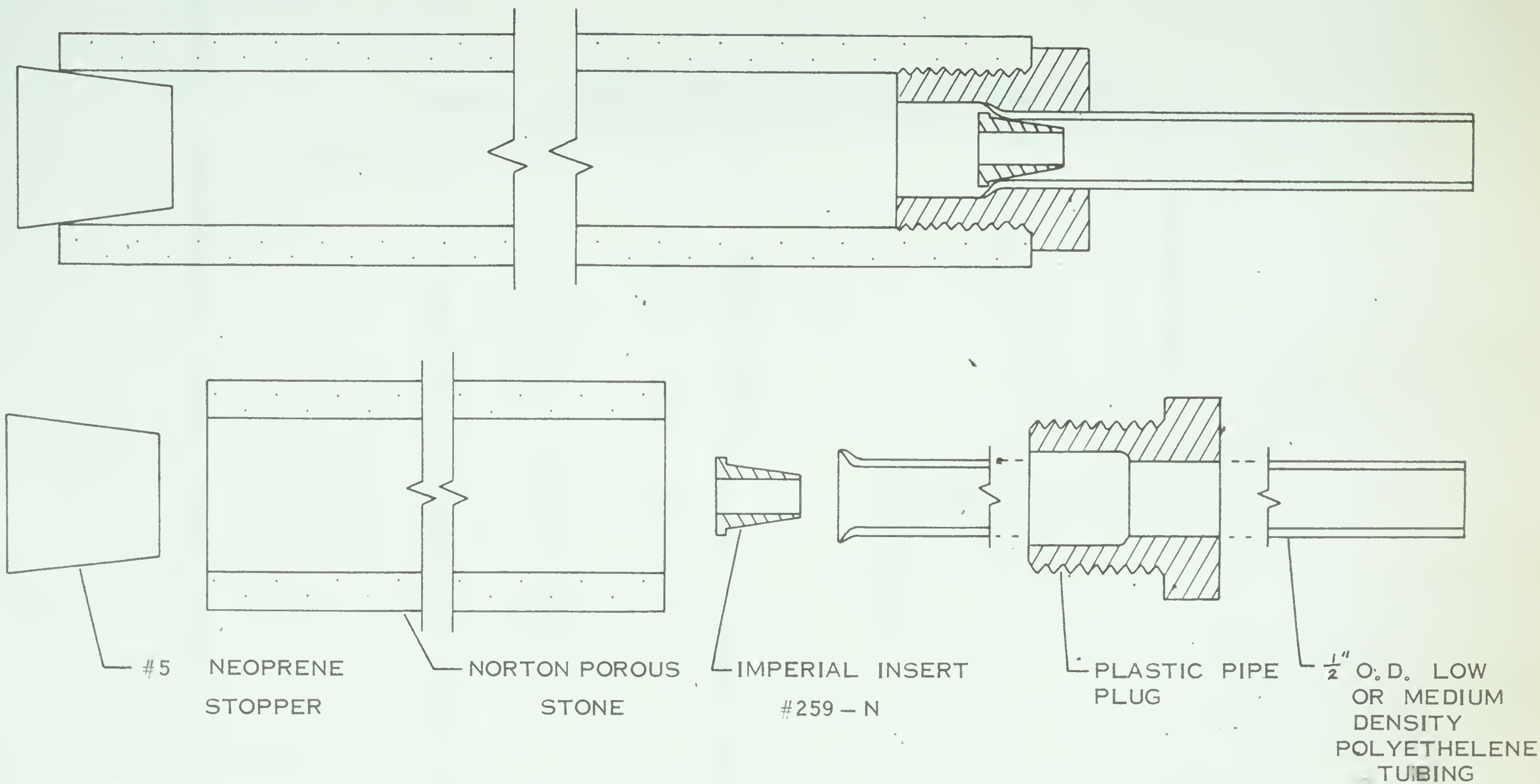


FIGURE 2.7 CASAGRANDE TYPE PIEZOMETER [AFTER P.F.R.A.]

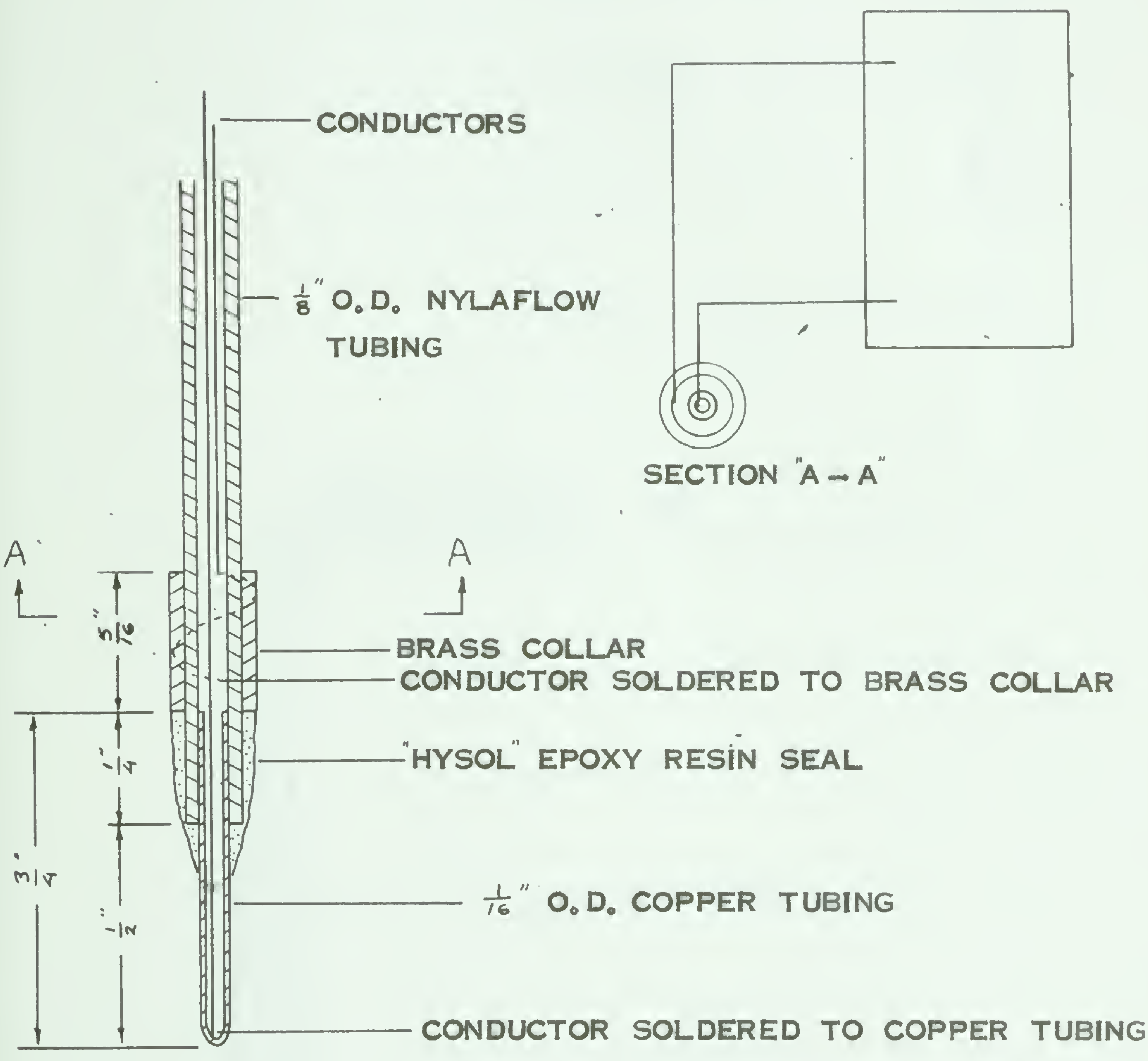


FIGURE 2.8 PIEZOMETER PROBE [AFTER BOZOZUK]

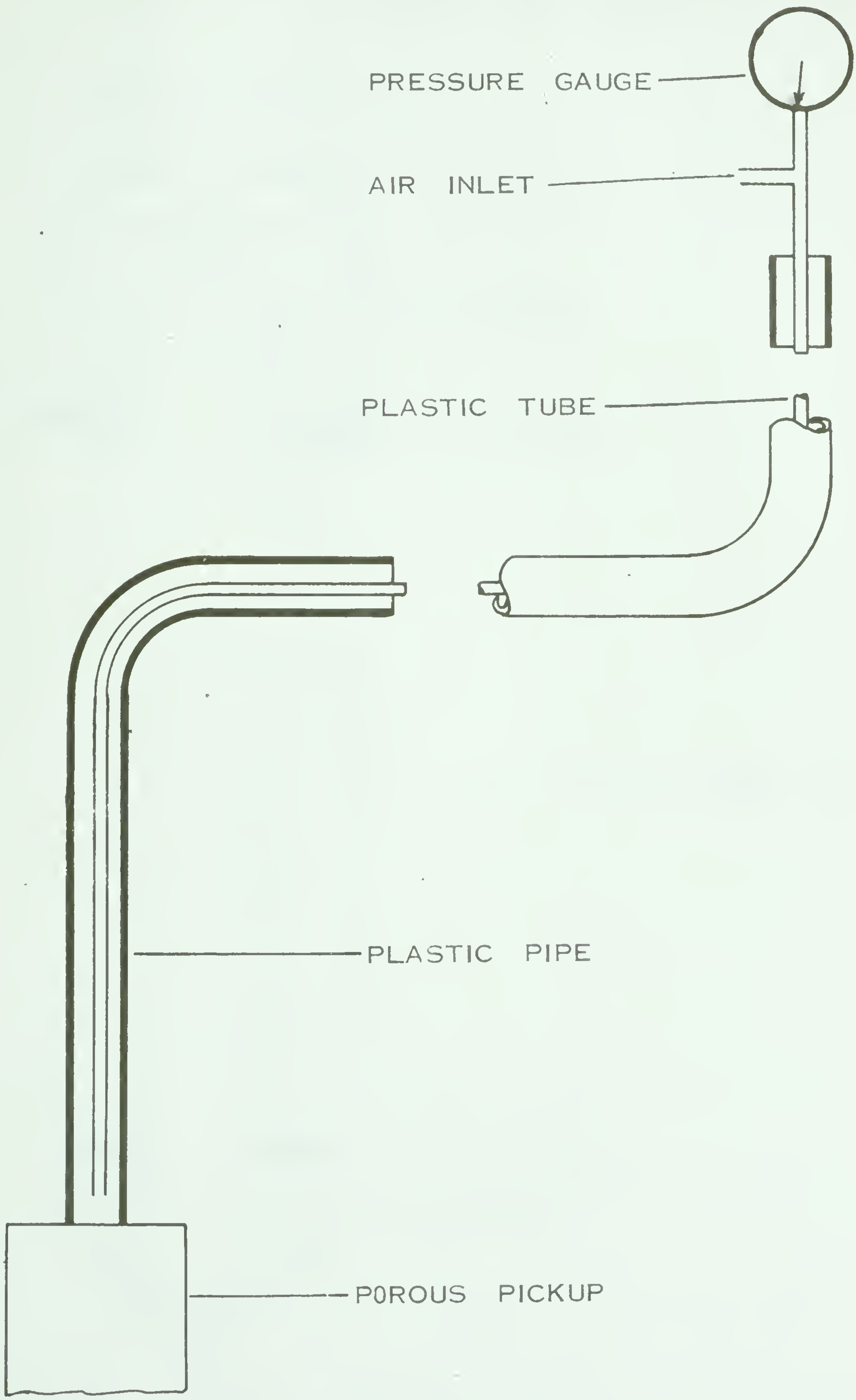


FIGURE 2.9 MODIFIED CASAGRANDE TYPE PIEZOMETER
[AFTER DE LUCCIA, 1958]

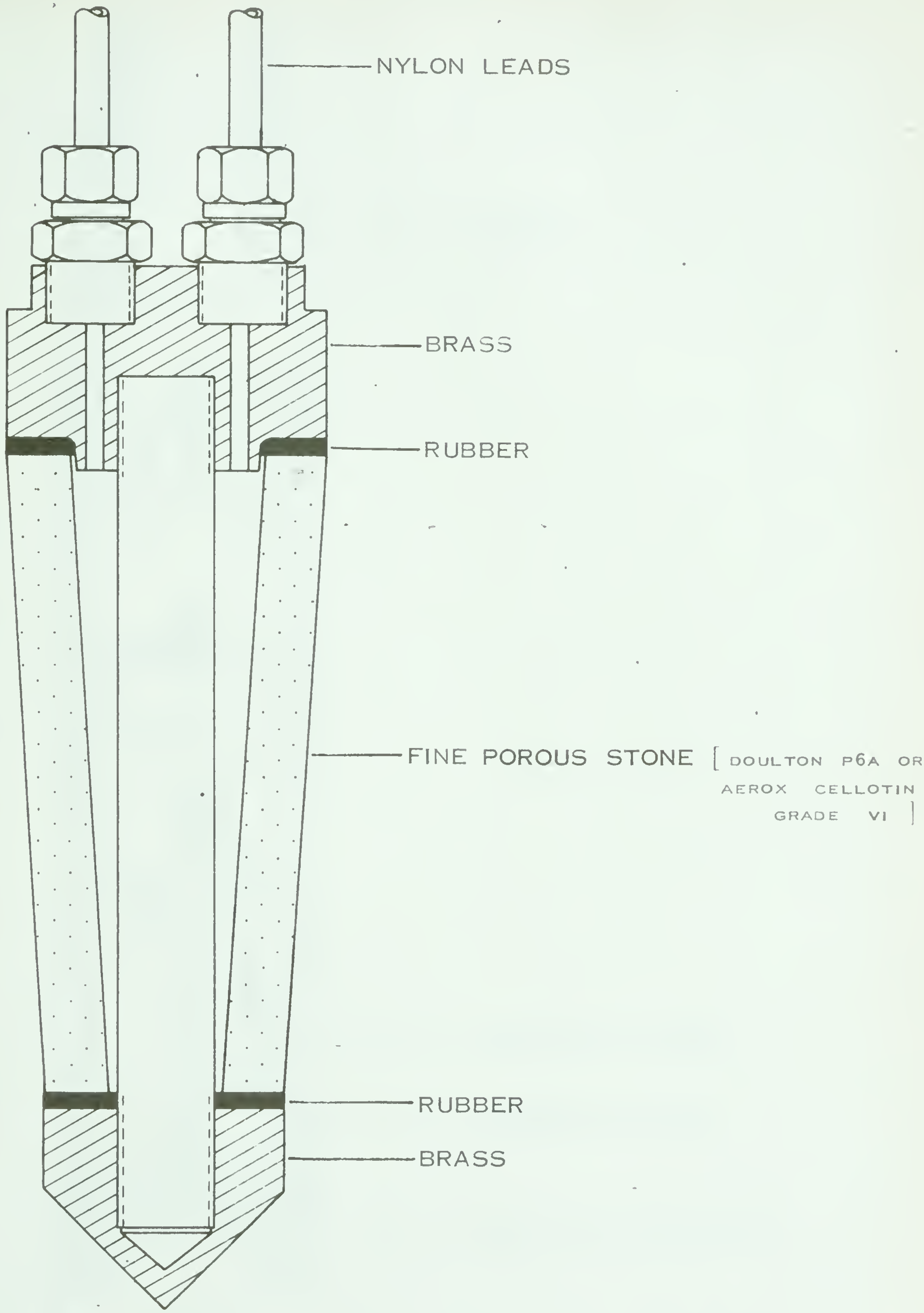


FIGURE 2.10 IMPERIAL COLLEGE [BISHOP] TYPE PIEZOMETER
[AFTER BISHOP, KENNARD, PENMAN, 1961]

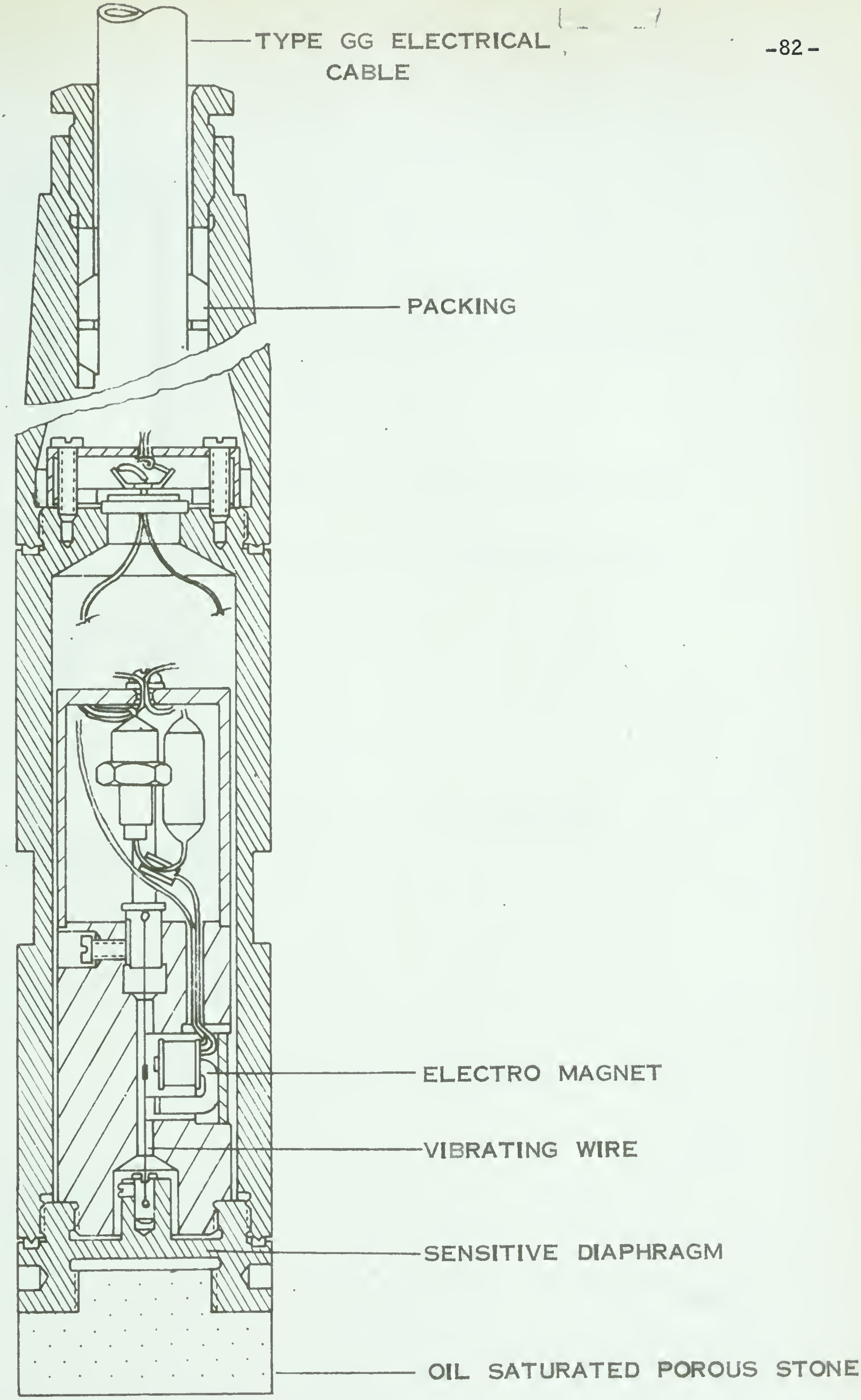


FIGURE 2.11 MAIHAK MDS-75 PORE WATER PRESSURE TRANSMITTER

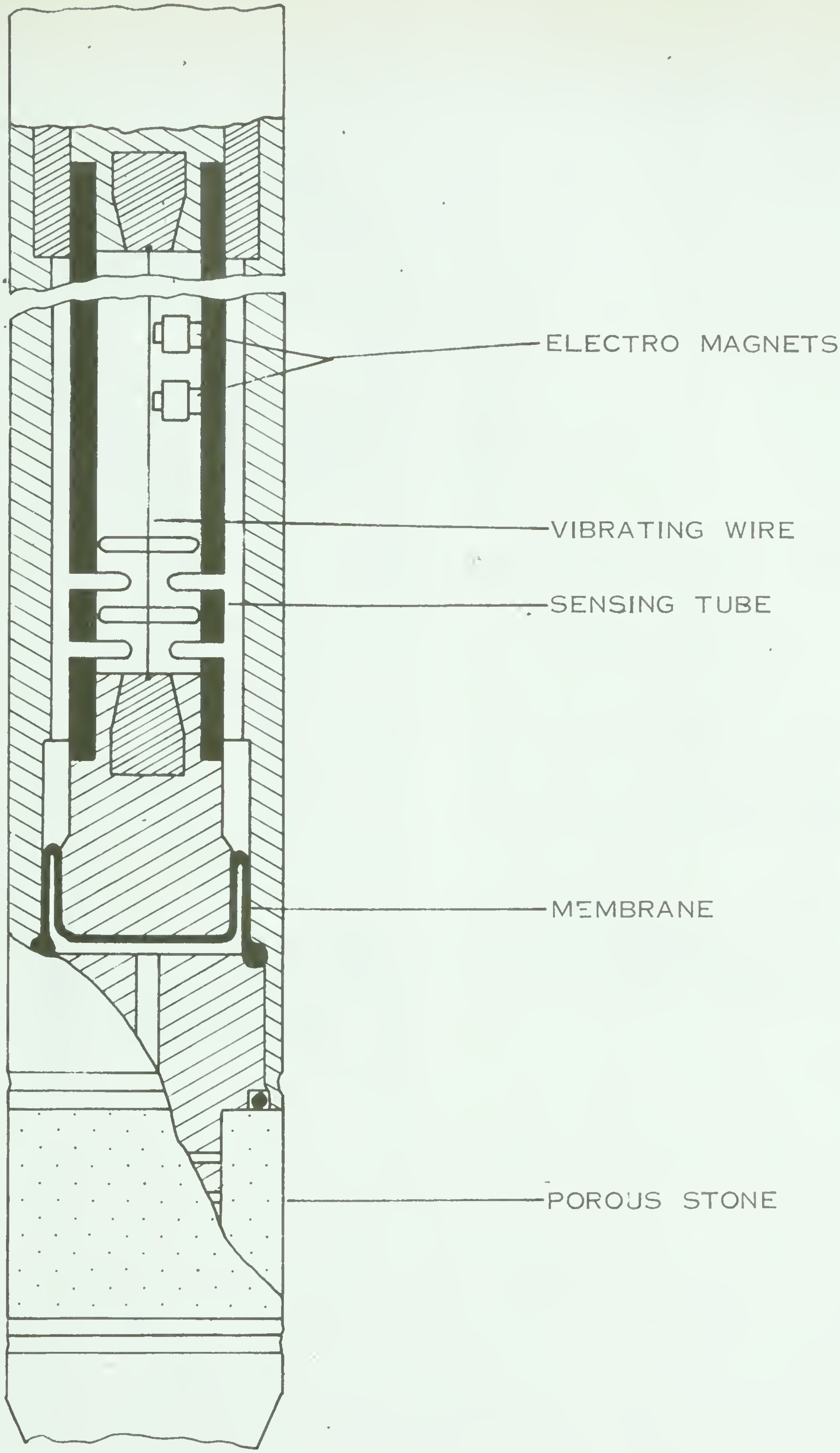


FIGURE 2.12 TELEMAC PIEZOMETER

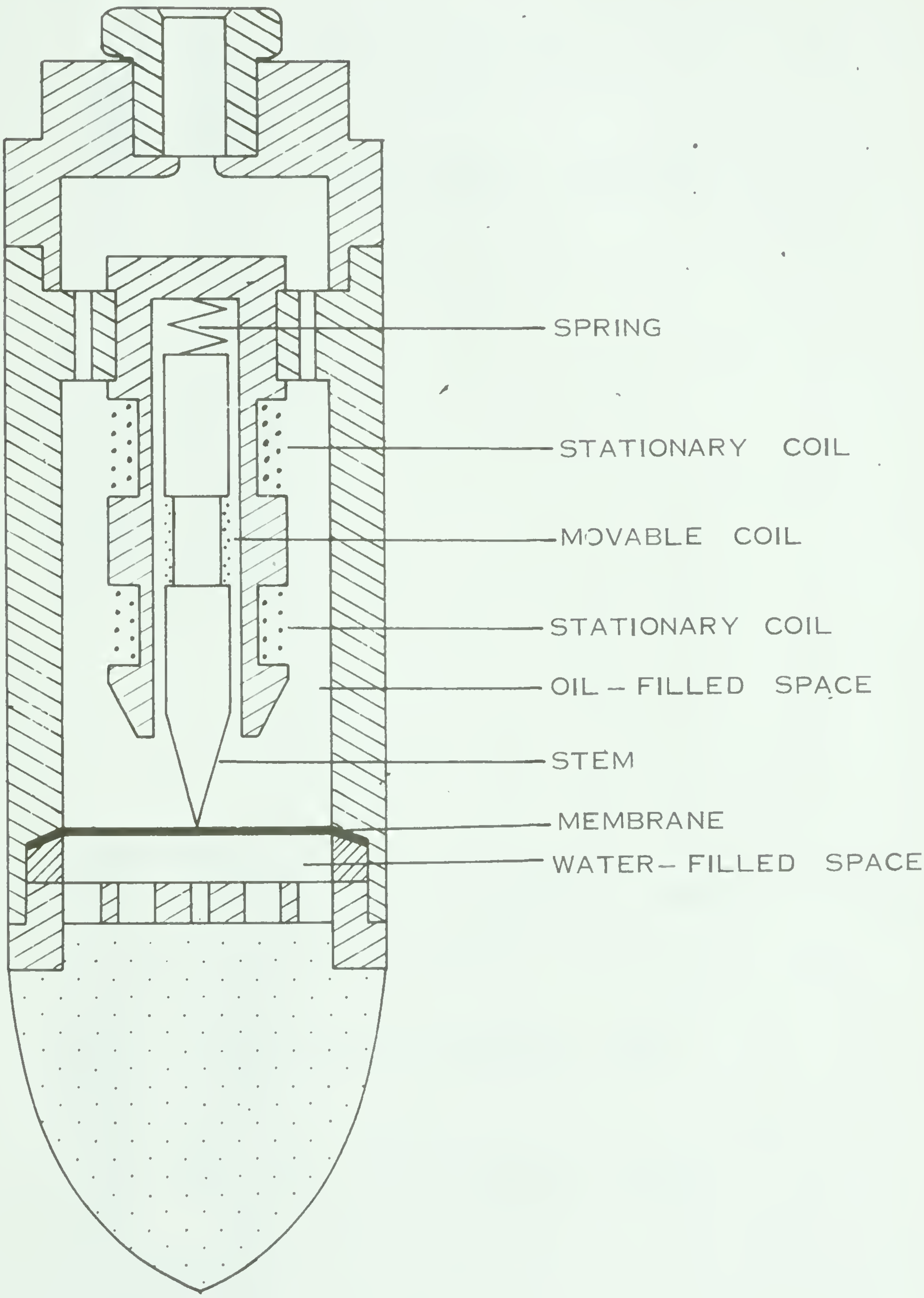


FIGURE 2.13 ELECTRICAL PORE WATER PRESSURE CELL [AFTER VOURINEN, 1957]

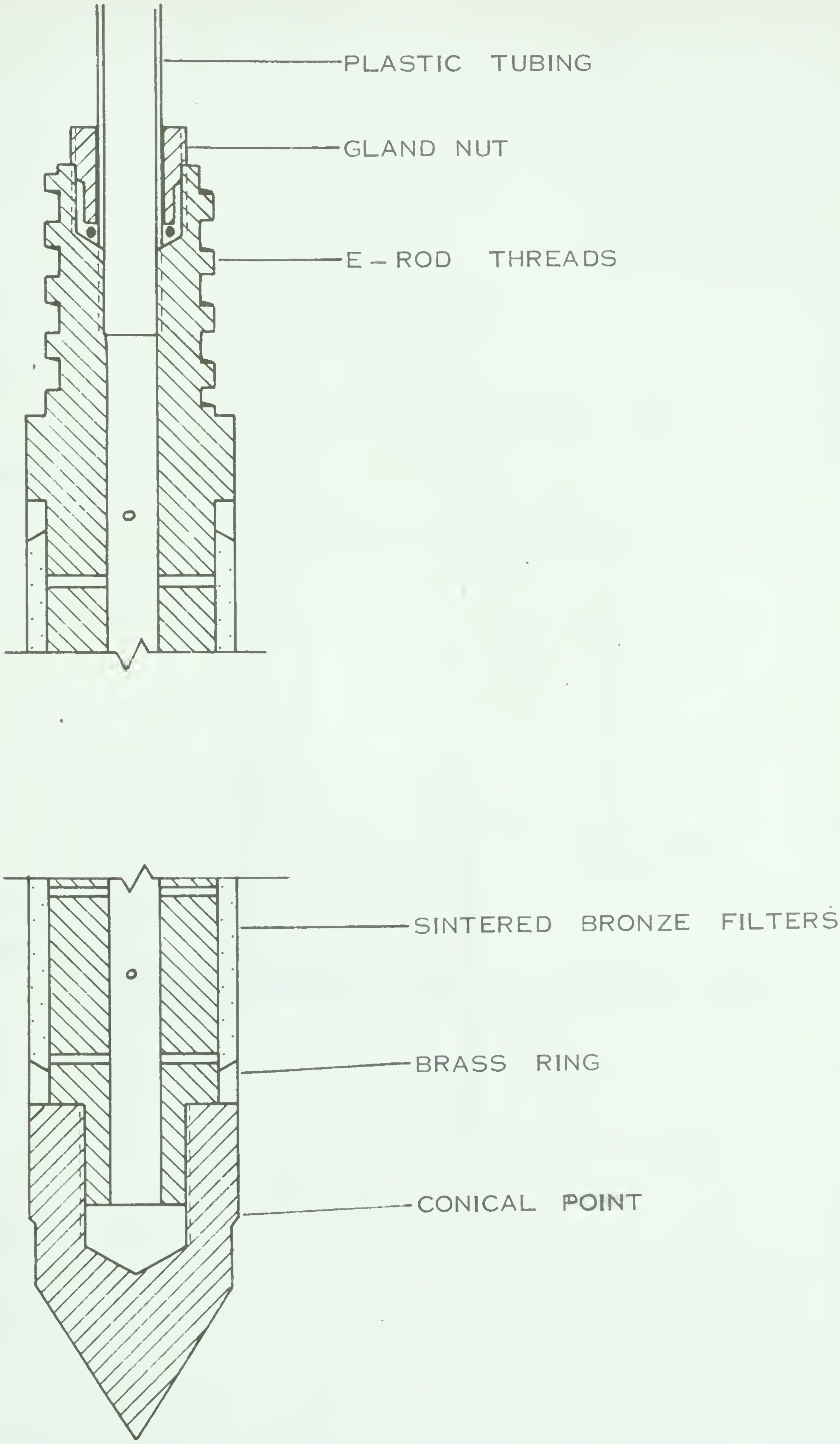


FIGURE 2.14 GEONOR PIEZOMETER

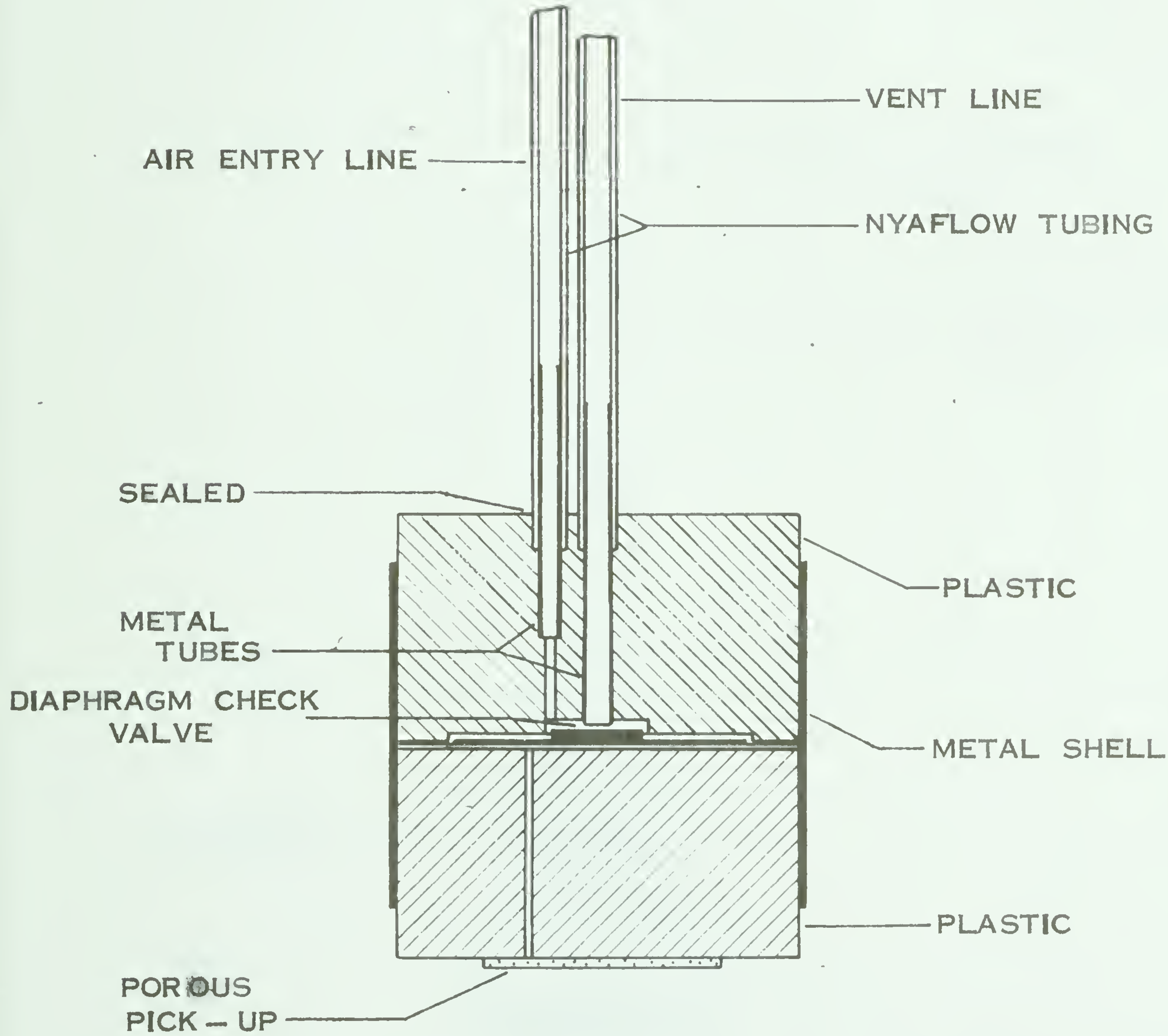


FIGURE 2.15 WARLAM PIEZOMETER

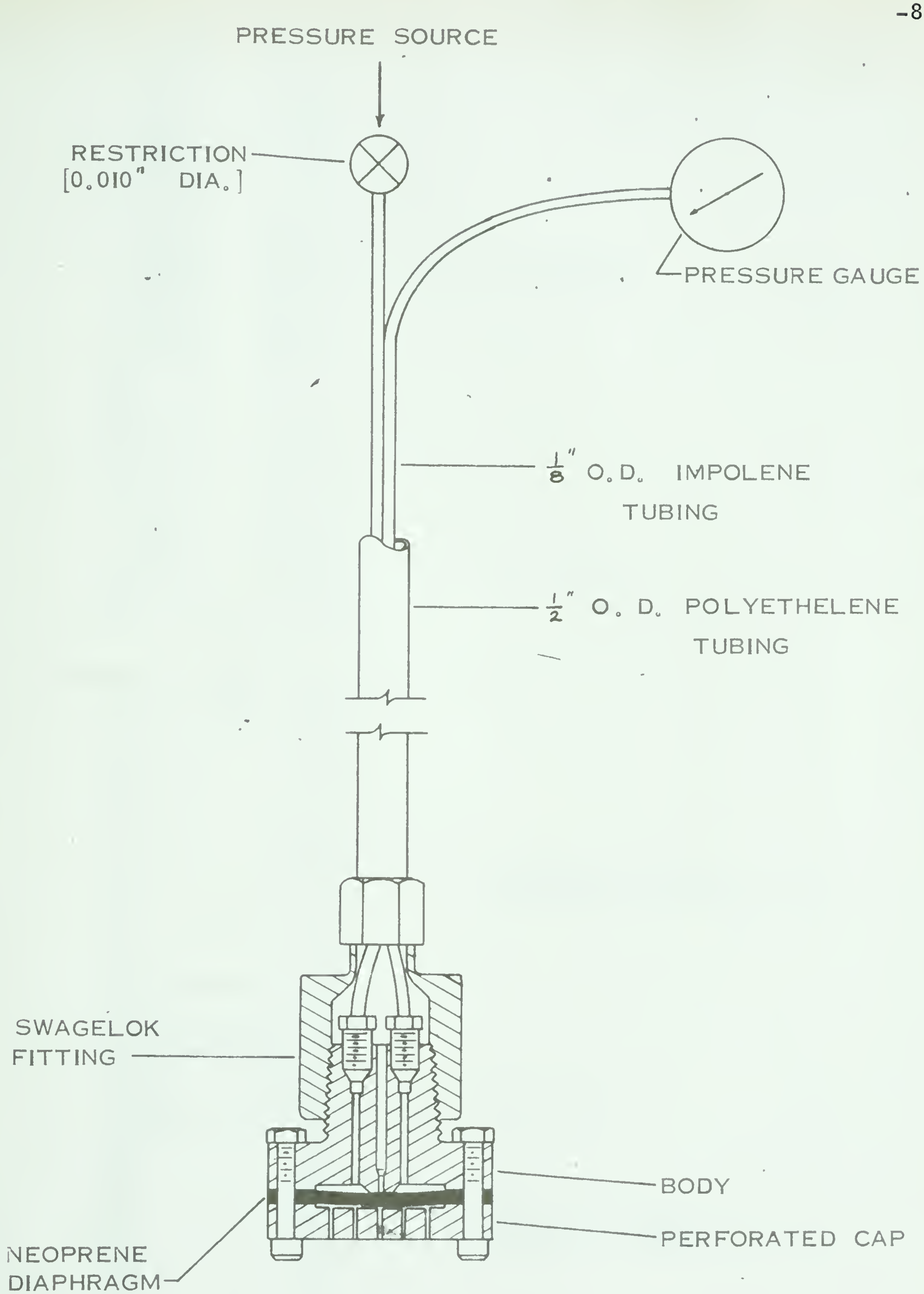


FIGURE 2.16 DAMES AND MOORE AIR OPERATED DIAPHRAGM PIEZOMETER

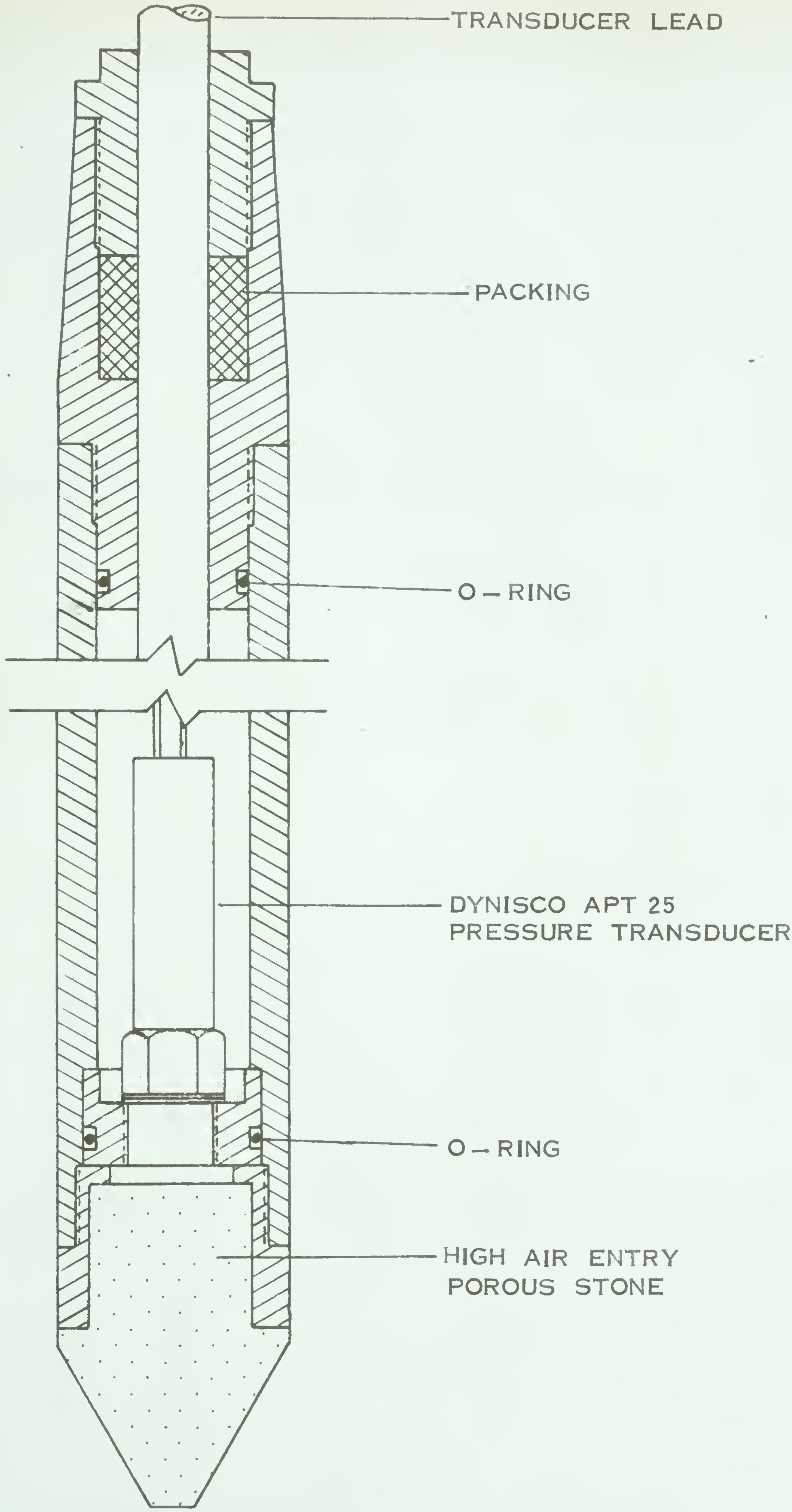


FIGURE 2. 17 U. of A. – G. S. C. PIEZOMETER

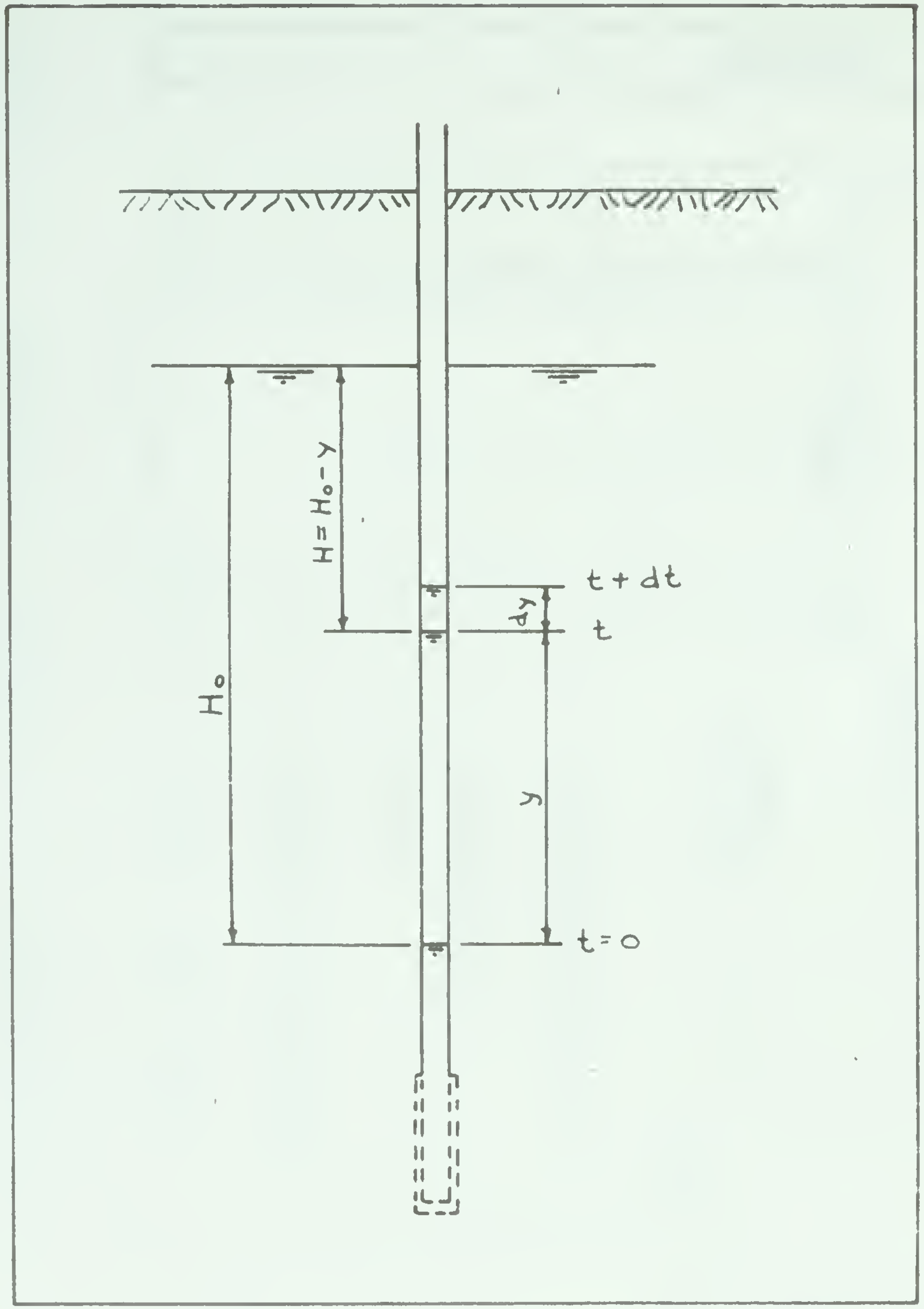


FIGURE 3.1 CONDITIONS ASSUMED FOR HVORSLEV'S THEORY [AFTER HVORSLEV 1951]

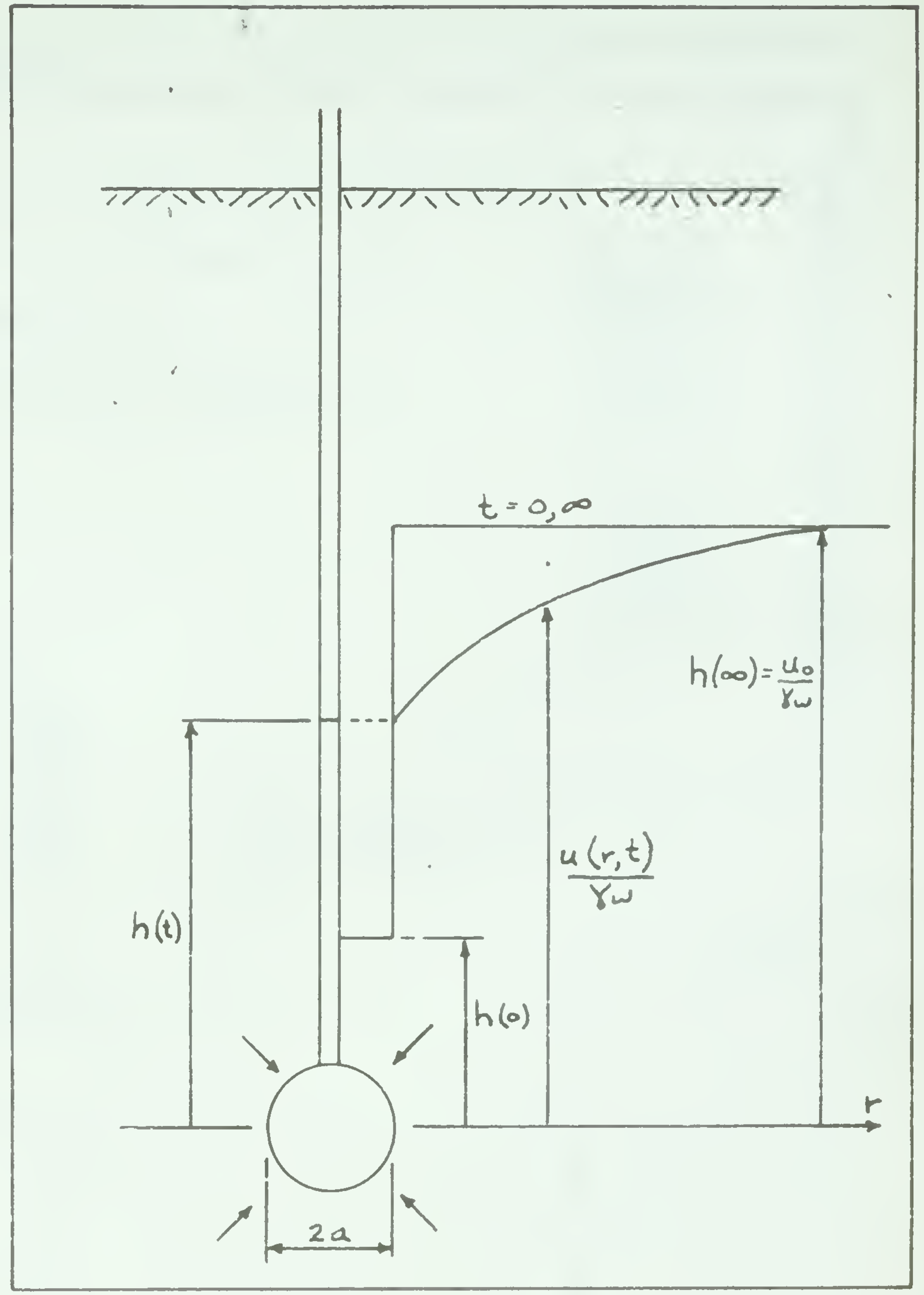


FIGURE 3.2 PORE PRESSURE DISTRIBUTION AROUND AN IDEAL PIEZOMETER TIP [AFTER GIBSON 1963]

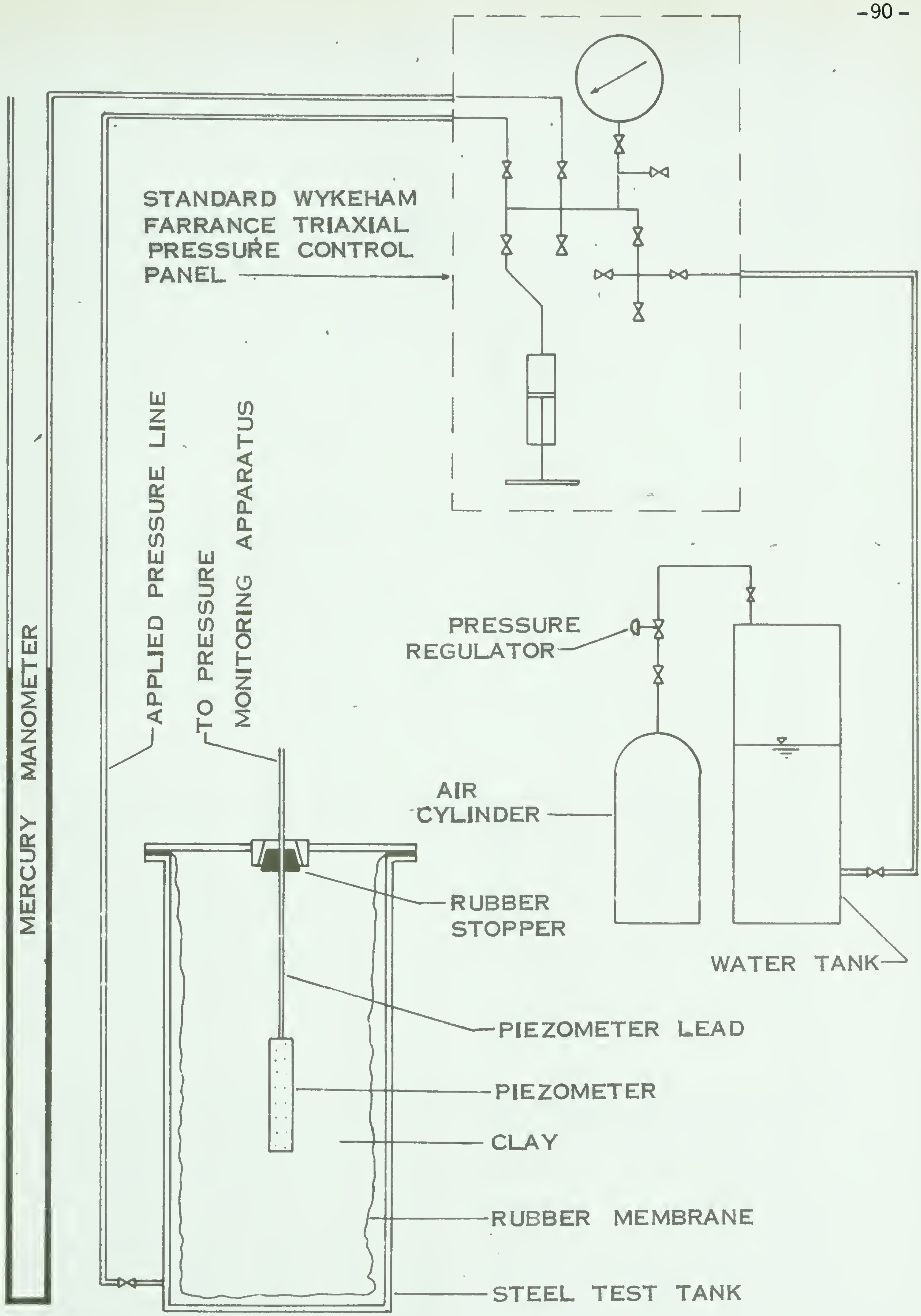


FIGURE 4.1 SCHEMATIC DIAGRAM OF PIEZOMETER TESTING APPARATUS

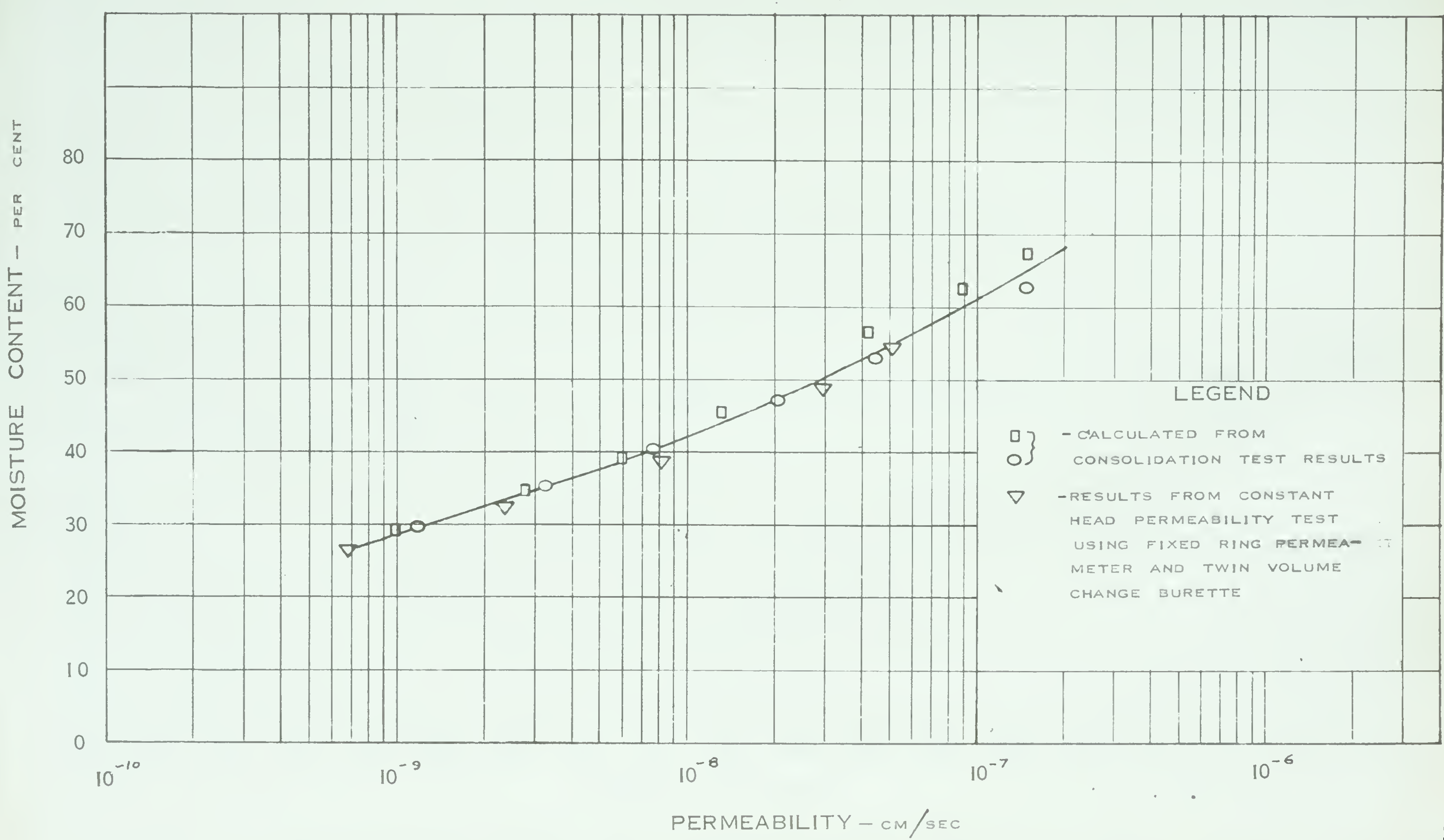


FIGURE 4.2 MOISTURE CONTENT vs PERMEABILITY [REMOLDED LAKE EDMONTON CLAY]

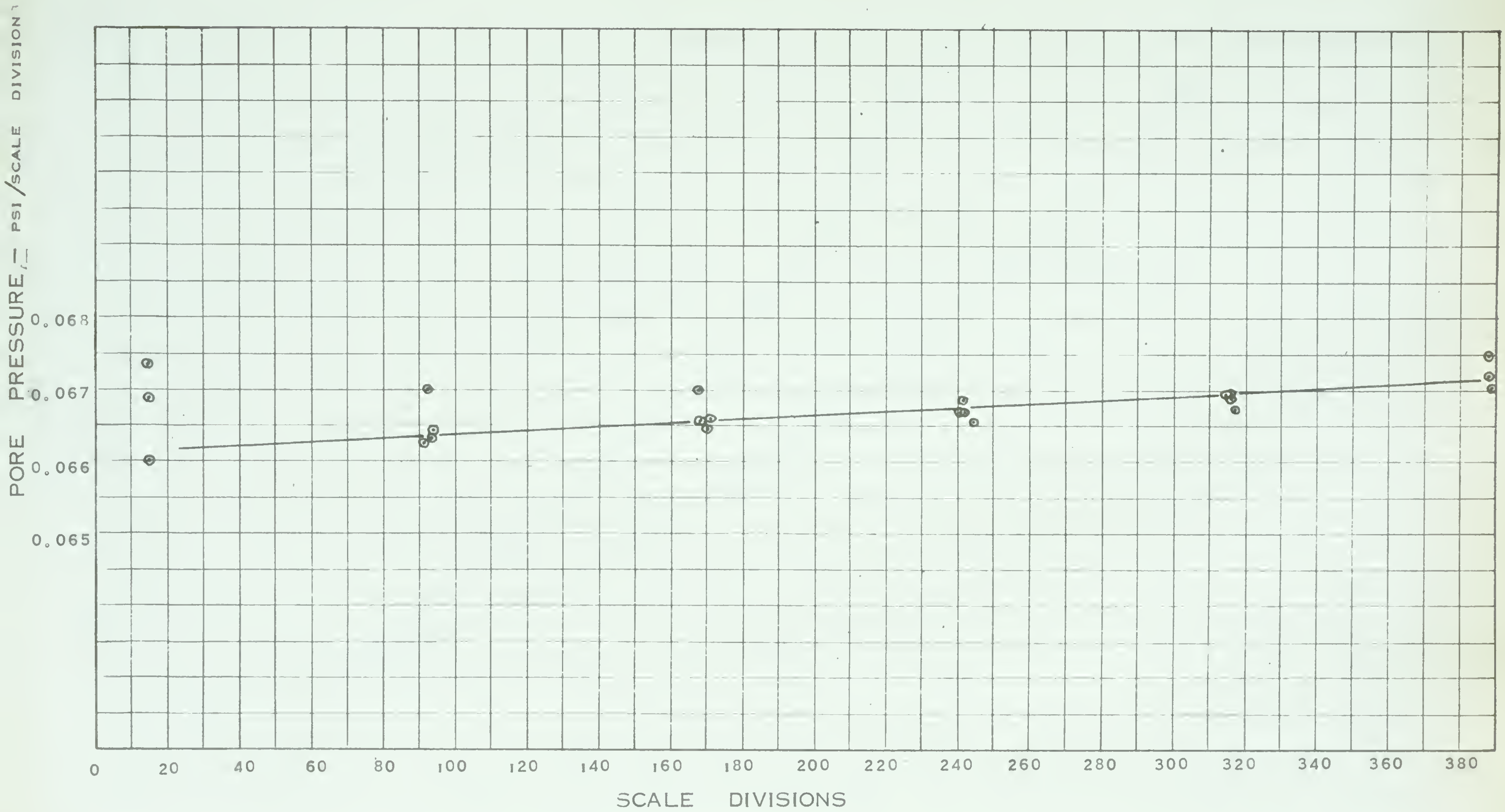


FIGURE 4.3 CALIBRATION CURVE FOR MAIHAK MDS75, SERIAL NO. 27817 WITH MDS-3, SERIAL NO. 39660

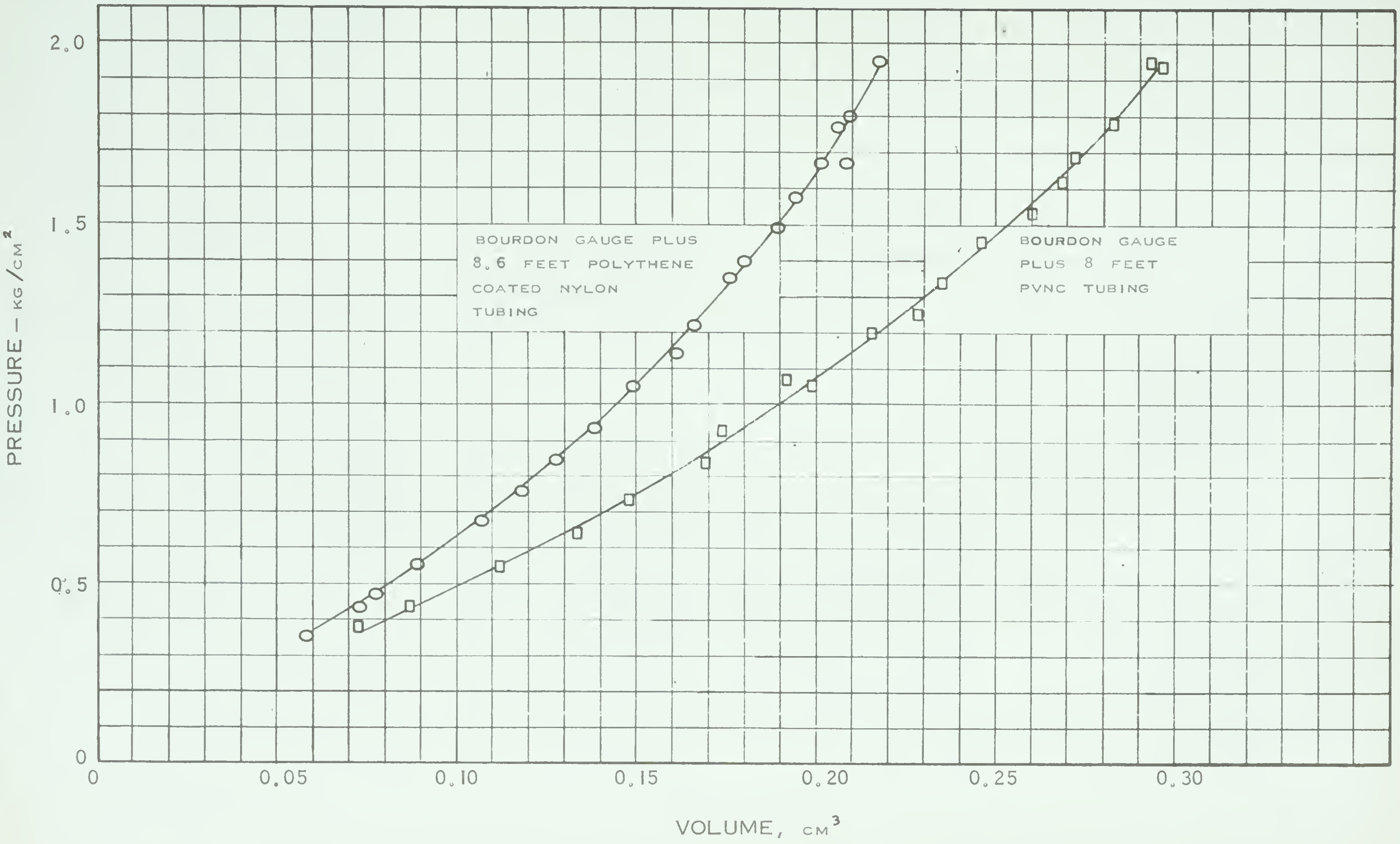


FIGURE 4.4 VOLUME vs PRESSURE RELATIONSHIPS FOR BOURDON GAUGE PLUS TUBING

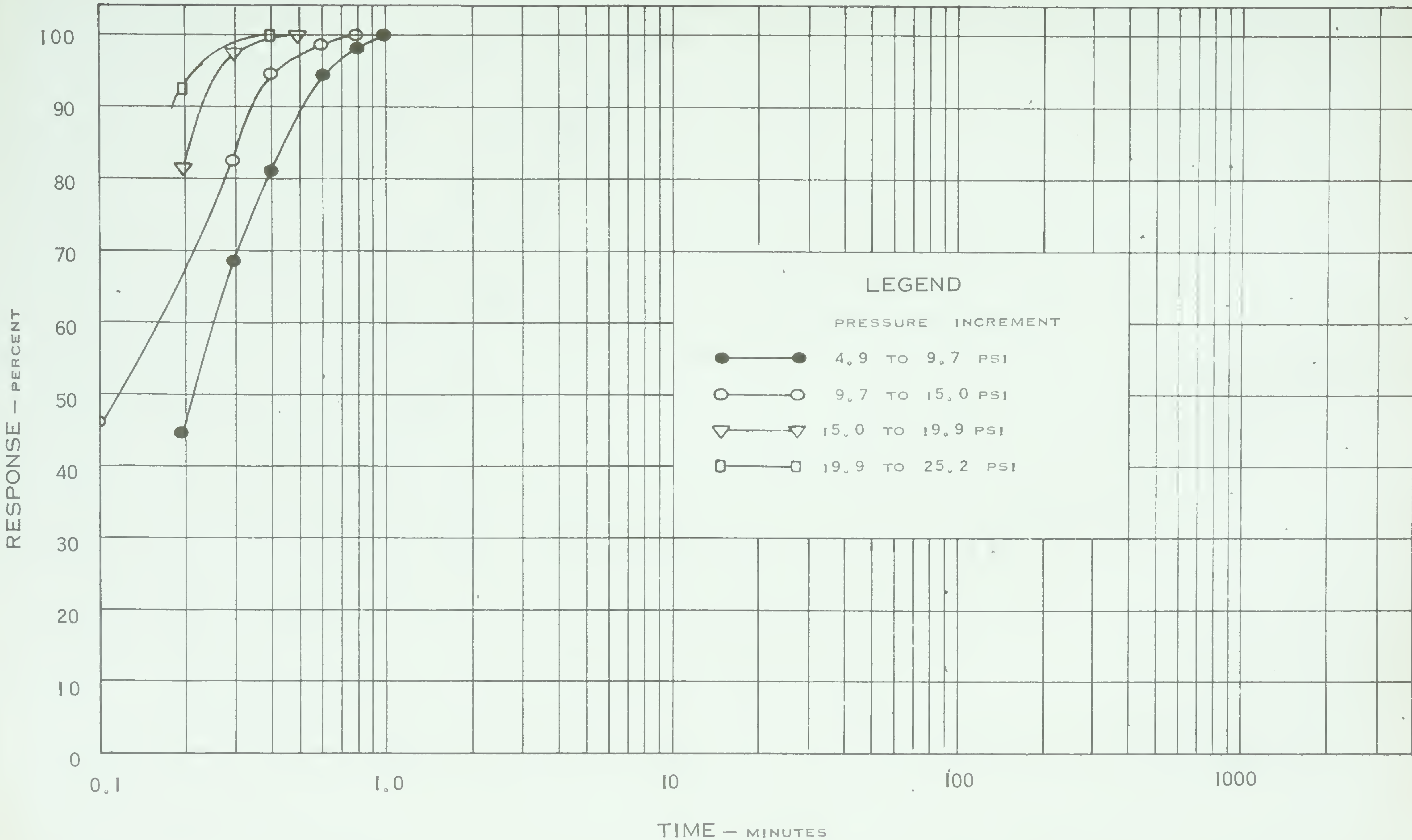


FIGURE 5.1 RESPONSE vs TIME, MAIHAK PIEZOMETER

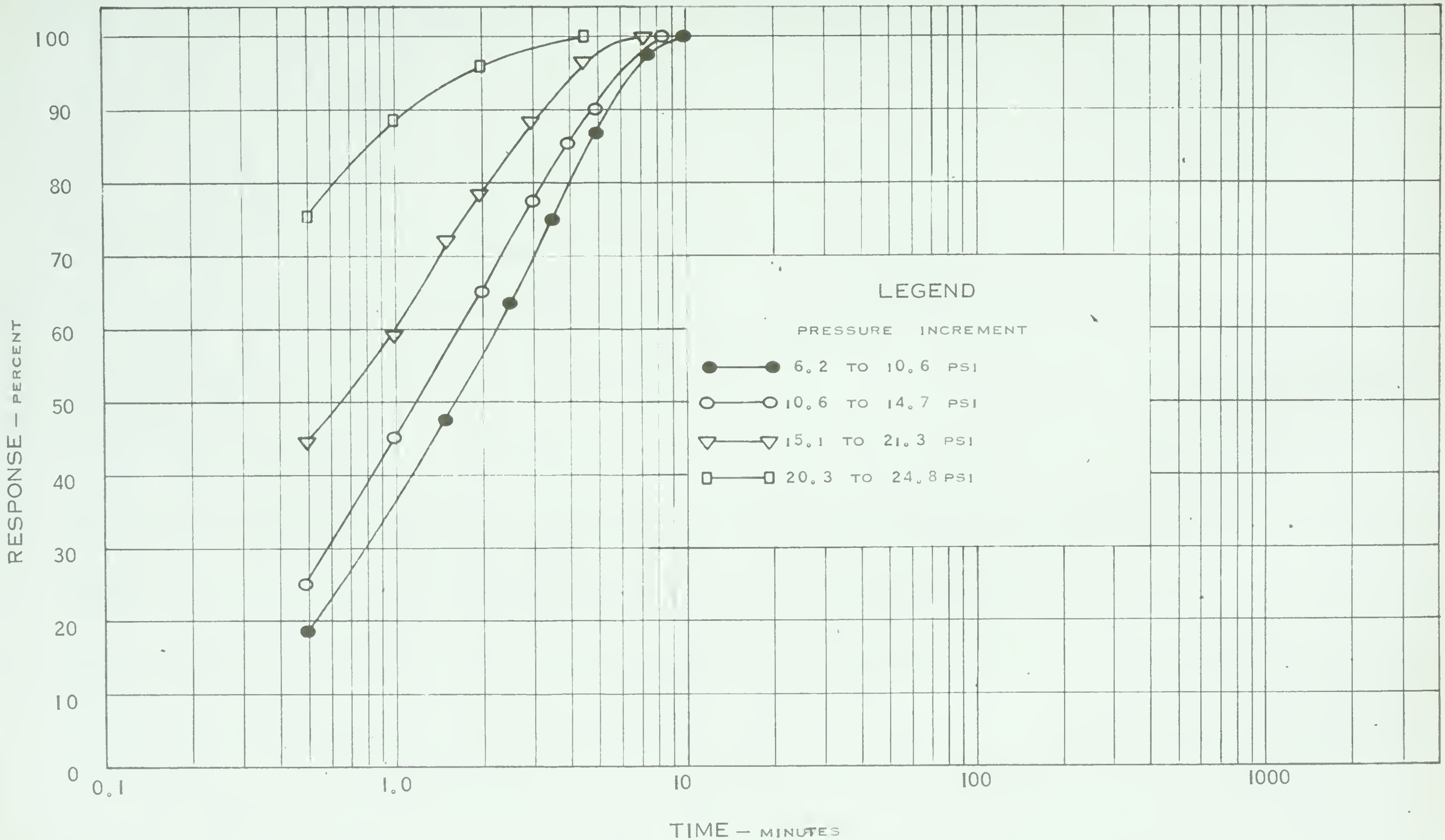


FIGURE 5.2 RESPONSE vs TIME, WARLAM PIEZOMETER

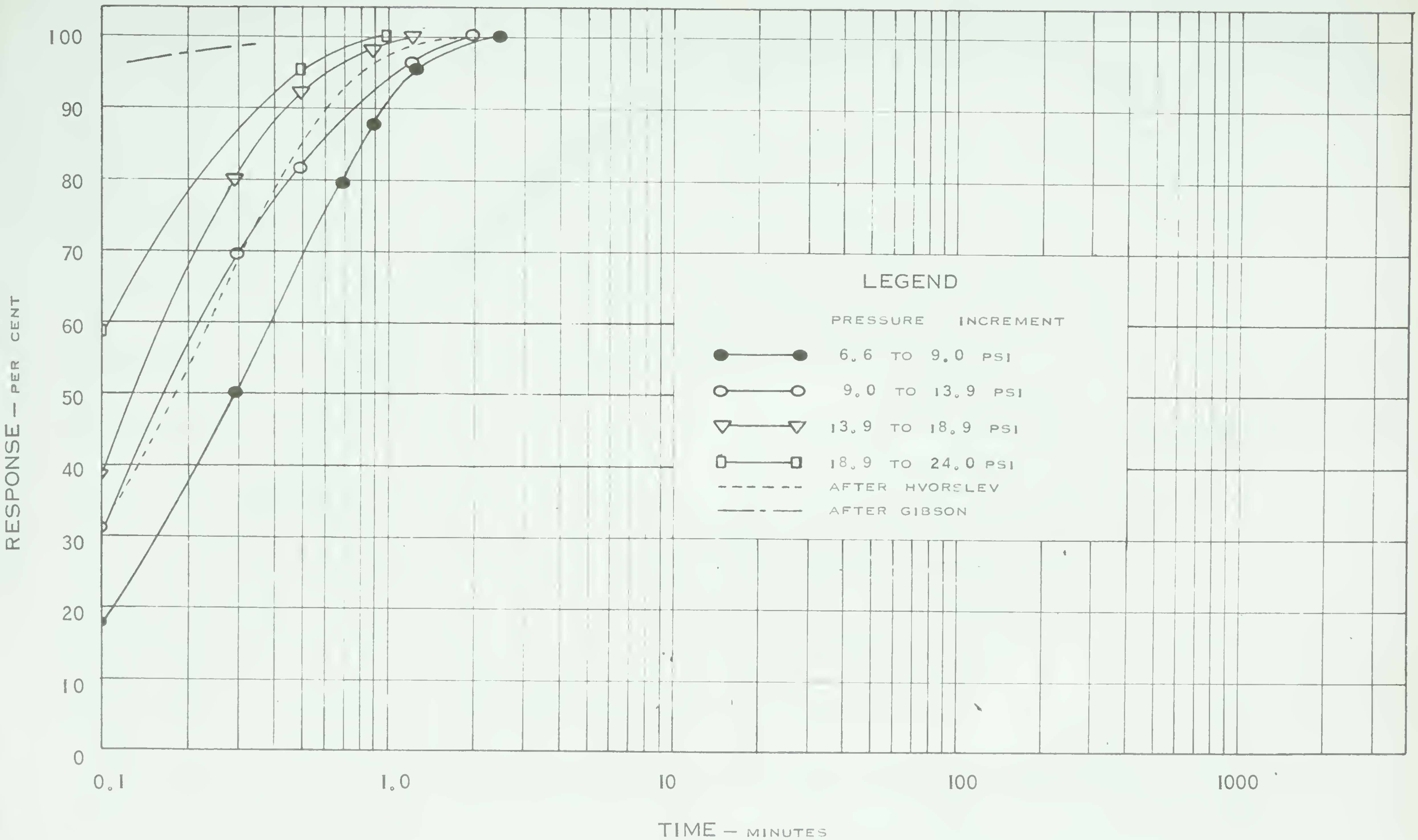


FIGURE 5.3 RESPONSE vs TIME, BISHOP PIEZOMETER [WITH BOURDON . GAUGE]

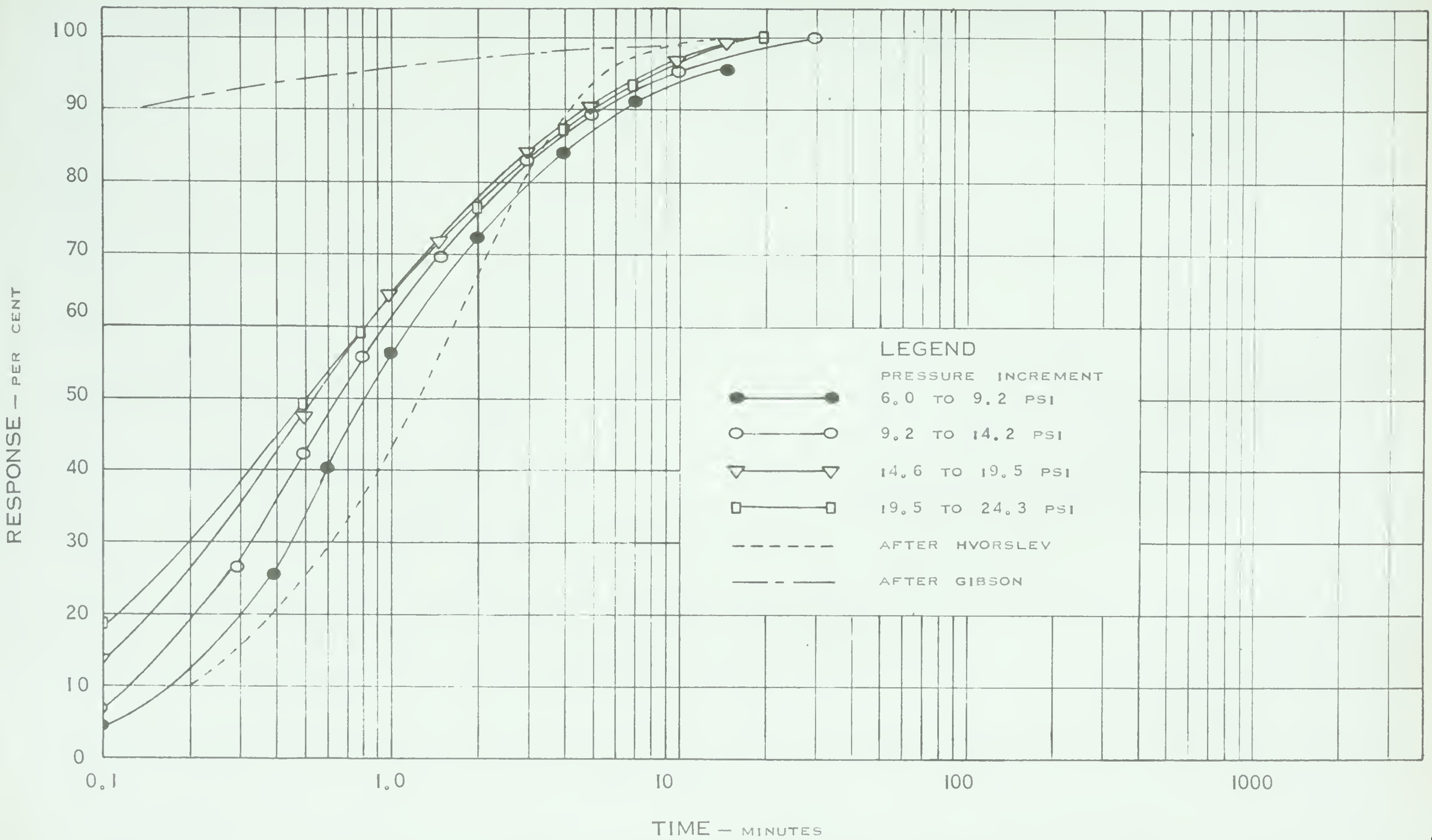


FIGURE 5.4 RESPONSE vs TIME, BISHOP PIEZOMETER [WITH MANOMETER]

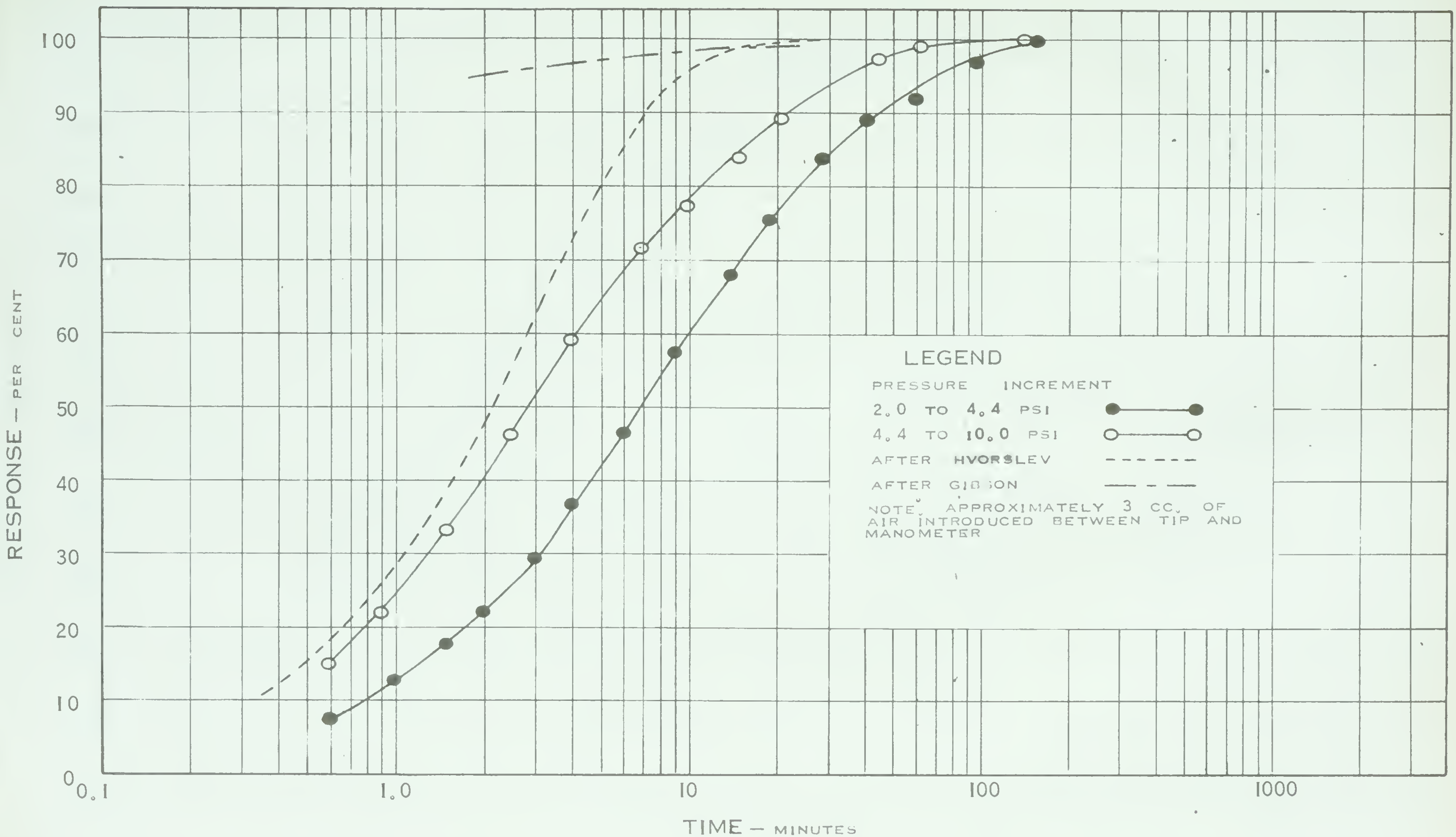


FIGURE 5.5 RESPONSE vs TIME, BISHOP PIEZOMETER [WITH MANOMETER]

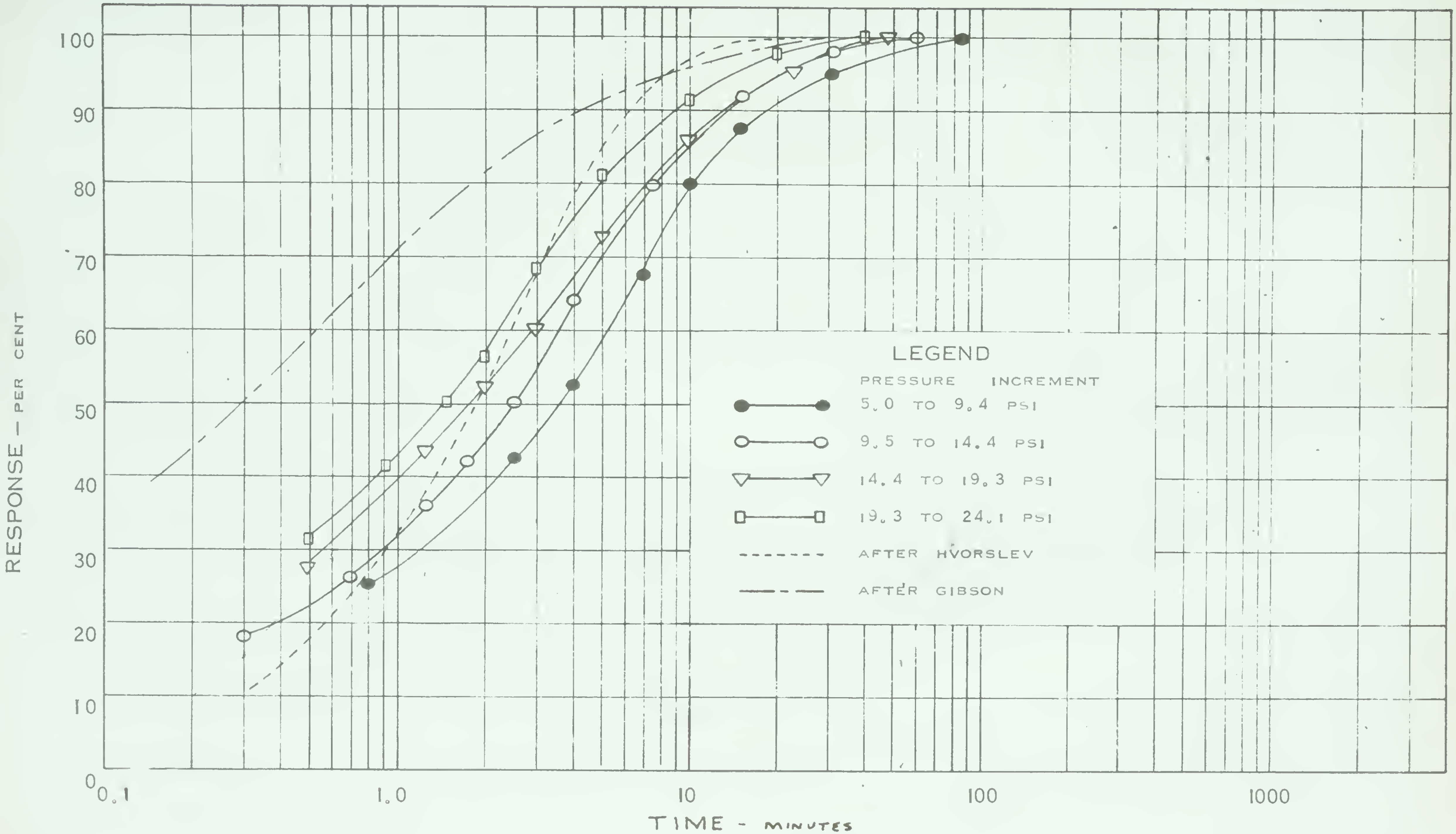


FIGURE 5.6 RESPONSE vs TIME; U.S.B.R. FOUNDATION PIEZOMETER [WITH BOURDON GAUGE]

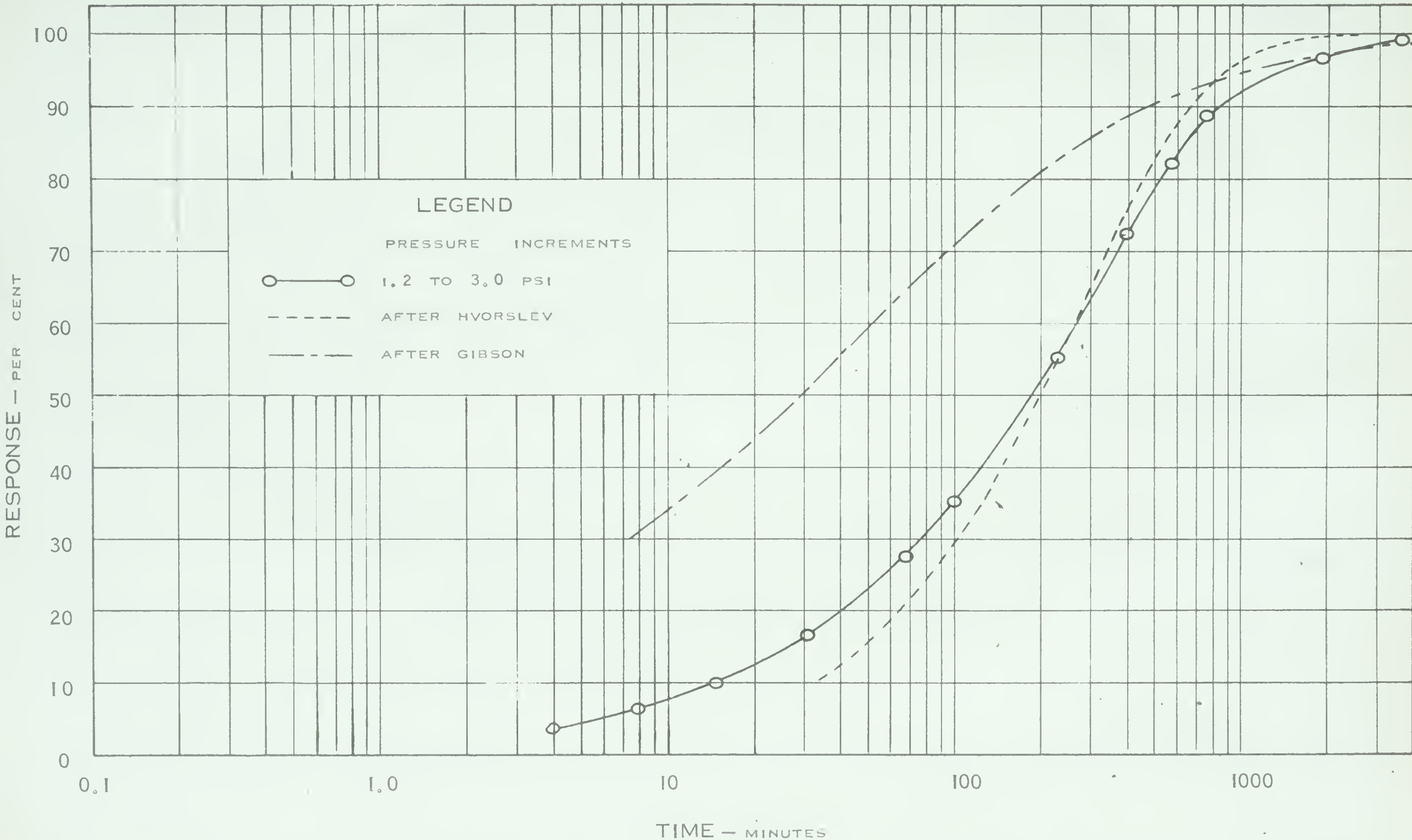


FIGURE 5.7 RESPONSE vs TIME, GEONOR PIEZOMETER

FIGURE 5.8 SUMMARY OF TYPICAL RESPONSE TEST RESULTS

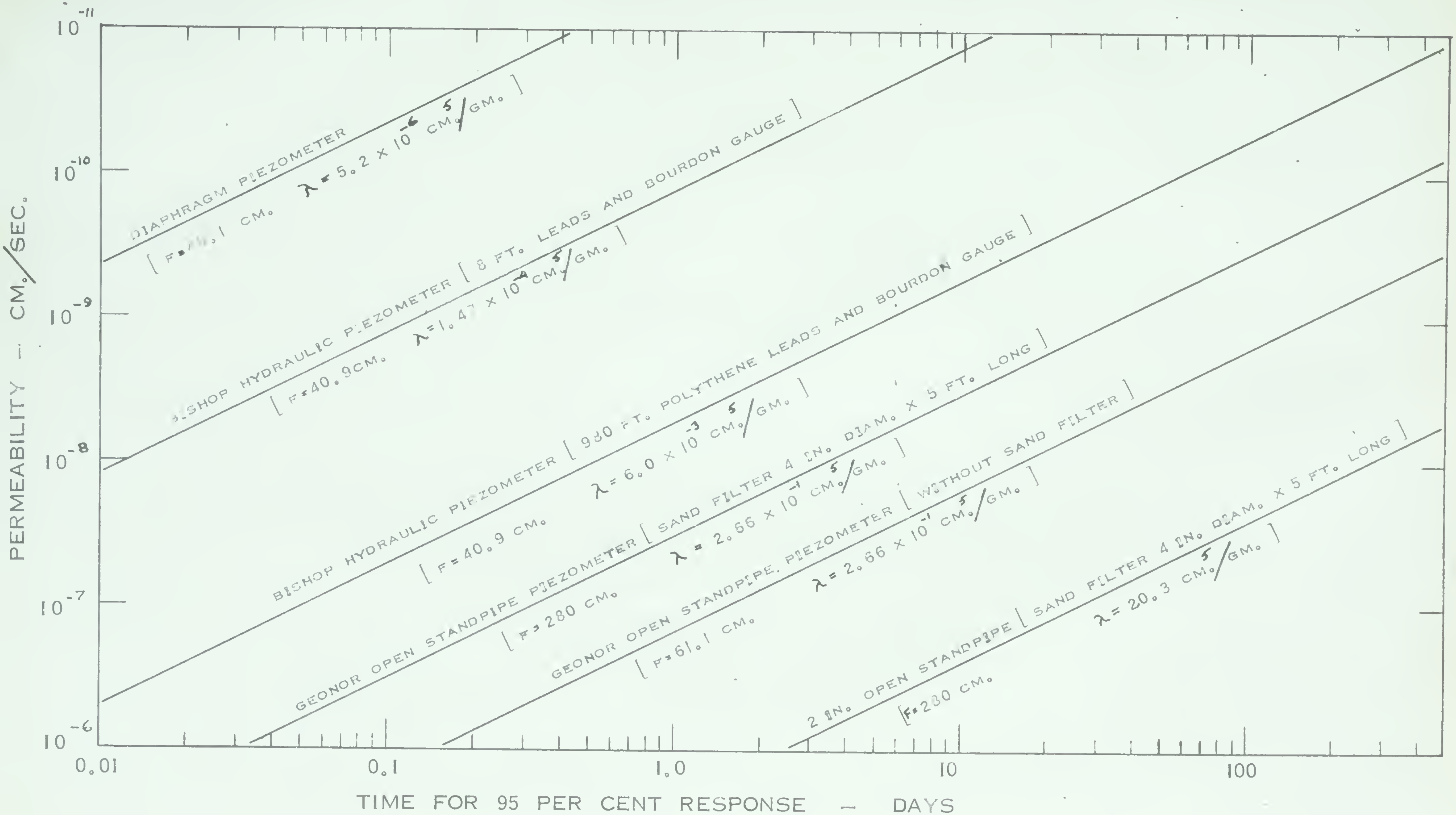


FIGURE 5.9 THEORETICAL TIME FOR 95 PER CENT RESPONSE vs PERMEABILITY

[AFTER HVORSLEY]